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A STUDY ON THE CONTROL OF GROUNDWATER HAZARD AT GAYS RIVER MINE

VOLUME IV

UNDERGROUND DEWATERING AND STABILITY CONSIDERATIONS

by: Golder Associates Ltd.

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CONSULTING ENGINEERS

REPORT ON

UNDERGROUND DEWATERING AND STABILITY CONSIDERATIONS GAYS RIVER, NOVA SCOTIA

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1.0 INTRODUCTION

This report has been prepared as a summary compilation of technical memoranda and factual data related to dewatering of the ore zone area at the Gays River Mine, Nova Scotia (Figure 1). The report is based on data collected during three site visits underground over the period May to June 1991.

1.1 Background

The report assumes the reader's familiarity with the following background documents:-

Esso Mineral Resources Ltd., (D.M. McKee & P.J. Hannon)	- 1982 -	The Hydrogeological Environment of the Gays River Mine
Jacques Whitford & Associates Ltd.,	- 1990 -	Gays River Overburden Stability Investigation
Western Mining Corporation (G. Meyer)	- 1991 -	Gays River Mine Dewatering and Water Inflow Prevention
Golder Associates Ltd.,	- 1991 -	Overburden Hydrogeological Investigation Report 911-1726-1

These reports describe the general hydrogeological conditions at the site and outline the general geometry and geology of the area of mining development (Figures 2 and 3).

Geologically, six main units can be recognized at the site:

- Recent Alluvium
- Pleistocene deposits, tills and glacio-fluvial sands
- Cretaceous Trench Infill (consolidated clayey sands)
- Carboniferous, Mississipian age, Windsor Group, Evaporitic Rocks (gypsum and anhydrite)
- Carbonate Reef Rocks (limestones, dolomitized limestones, basal breccias etc.), also of Carboniferous (Mississipian age)
- Ordovician, Goldenville quartzites of the Meguma Group

Currently, mining is being undertaken of the mineralized margin of the Windsor Group carbonate rocks adjacent to the Pleistocene and Cretaceous trench infill material (Figure 2). As this margin follows the foreshore edge of a relict Carboniferous coralline reef, in plan, the mine workings are of complex shape, essentially mirroring the steep valleys and rolling fore-reef topography (ref. Figure 3).

Nevertheless, despite the planimetric complexity, in section, the characteristics of the reef margin and trench infill are similar right across the mine site area.

1.2 Scope of Work

The scope of work for this study was formulated to address one of the principal needs identified at a technical review meeting held under the auspices of CANMET on May 13th and 14th, 1991. The study, which has been carried out from June to September 1991 has essentially involved assessment of underground inflow magnitudes and pressures and evaluation of rock mass characteristics to provide advice to the mine for dewatering purposes.

Data acquisition was principally undertaken on three underground visits. These were made on the following dates:

- May 13th, 1991
- June 10-12th, 1991 and
- June 28th 1991.

On the basis of information collected during these visits and data provided in follow-up discussions with mine personnel, numerous memoranda were issued to assist mine staff towards improving dewatering effectiveness.

Copies of the original memoranda, as issued over the period from June to August relating to groundwater inflow conditions, are contained in Appendix A, in chronological order complete with the original recommendations given to the mine on the methodology for the testing procedures. In Appendix B, the tabulated pressure and flow data collected from various monitoring installations installed during the second site visit are presented, while in Appendix C

the memorandum issued on June 20th, 1991 relating to overburden stability considerations and piezometer installation within the trench infill material is presented. Data on the types of suitable piezometers recommended at that time for inclusion within the trench infill material are also included in Appendix C.

As this report is simply a compilation document, collecting together the various memoranda issued to assist mine staff focus their dewatering effort, the report does not purport to provide a comprehensive assessment of the underground flow regime, this is outside both the scope of this report and the original terms of reference. Rather, some guidelines are provided in the main text and in the appendices. Mostly, these guidelines are of a practical nature, relating to various technical issues requiring solution if effective dewatering of the Carboniferous evaporite deposits and the younger trench infill material and its margins is to be accomplished and maintained.

2.0 INITIAL OBSERVATIONS

Following the technical review meeting held on May 13th and 14th, 1991, and an initial underground inspection of the general western section of the mine; it was noted that ongoing dewatering had been fraught with problems due to hole blockages, excessive fines washout (Plate 1) and even mudrush inflows when dewatering pressures rose uncontrollably (Plate 2). From the underground dewatering viewpoint, several technical issues were identified as critical to improving on previous dewatering successes, namely:

- Optimization of future drainhole locations,
- Rationalization of drainhole drilling methods and selection of appropriate screen types to avoid continuing washout of fines (thus hopefully ensuring stability of future drain holes and longevity of dewatering effectiveness),
- Development of an appropriate layout of monitoring points to aid understanding of dewatering progress and events, and
- Development of ideas to assist in prediction and control of hangingwall sloughing/inflow movements into the underground stoping operations.

On the basis of some preliminary analyses of material sampled from the trench zone, (see Figure 4 for grading curves and descriptive details) and based on inspection of mud-flow conditions underground (Plate 2), the recommendation was made, at and subsequent to the May review meeting, to start to install slotted stainless steel screens in all future drainhole locations (Note: a program of additional drilling was scheduled to begin in June using an Aardvark drill - ref. Figure 5).

During the second site visit underground (10-12th June, 1991) several Aardvark holes had already been drilled and drilling was still underway in the 808 X-cut (see location C - Figure 6). At this time, drilling and screen installation procedures were reviewed and several pressure monitoring points were installed across the entire width of the mining operation.

In total, ten points were installed over the period 10-12th June; two additional gauges being installed by mine personnel subsequently. The mine grid coordinates and elevations, referenced

to mine datum of each of these points are tabulated below:

Pressure Test Point		Estimated Mine Grid Coordinates (metres)		
Number	Location	Northing	Easting	Elevation
PT-1	118 Sump	6979.60	8533.00	410.00
PT-2	825 Ramp	6694.72	8123.80	441.04
PT-3	808 Sump	6758.50	8097.00	421.50
PT-4	848 Ramp	6801.00	8485.00	441.86
PT-5	851 Decline	6793.50	8587.17	428.68
PT-6	803 Sump	6692.19	8021.97	427.35
PT-7	101-D Drift	7209.20	8881.50	412.00
PT-8	119 Sump	7110.00	8570.70	428.00
PT-9	201 Drift	6990.93	8940.39	471.81
PT-10	118 Hole 26	6996.00	8541.50	407.50
PT-11	804 Hole	6560.00	7967.30	416.00
PT-12	804 Hole Ring 1, Hole 4	6663.60	7978.50	417.50

The location of each of the above listed monitoring points is shown on Figure 6.

Following the June review and evaluation of available data on flow and pressure behaviour, the following point form summary of technical points was presented to mine staff:

- 50 slot stainless steel screen seems appropriate for all the Aardvark drillholes as it seems to be able to deliver water flows desired without major loss of insitu material. Such screen will stop movement of material of sand size upwards and is sufficiently robust to prevent damage due to loose stone problems. *If and only if* significant amounts of fine material is coming through the screens, then as a worst case option, it may be necessary to go to telescoped finer slot screen inside the 50 slot installation although there may be a tendency for this to plug or cake-up. Flushing (using an air lance) is really the only effective solution, until the screens develop their own natural graded insitu gravel pack.

- Carry out gradings and Atterberg limits for several different types of trench material *and* also for the mud materials in 101D and on the drift invert at 851 Decline and 848 Ramp. Data acquisition may help understand timing and effectiveness of sand/gravel pack "development" in the insitu trench material around new Aardvark type holes - it may also help understanding of possible increases in trench material permeability with time - hence possible increases or migration in flow patterns long-term.

- Consider dewatering wells from surface into areas at *ends* of "drawdown cone" on strike of trench/gypsum contact.

- Spread out dewatering points to extend "curtain drainage" along the trench/gypsum/carbonate strike - target drilling to hit deep spots with high hydraulic conductivity and high existing pressure heads *first*.

- Develop scrolled-out longitudinal section along trench for plotting and monitoring pressure heads along deposit.

- Develop better understanding of pressure head data along trench - monitor PT holes daily while Aardvark drilling underway - attempt to integrate data with elevation head measurements from surface holes (ETW and DW series etc). This latter information may be complicated by mixed unit pressure head data from full column gravel packs/open hole conditions in some of the surface holes.

- Add additional underground pressure monitoring points in east end of mine as fill-in to holes gauged on 11/6/91. Consider monitoring at least one of the drain holes in 804 area now that the Aardvark drill will be moving to that area (ref. Area B - Figure 6).

- 101 D area (ref. Area H - Figure 6) - worthwhile to try to get good control on what's happening here. Try to put grout plugs on *all* holes in this area - have two holes pressure gauged - fastest flow hole and slowest flow hole. Turn off all valves, slowest flowing hole first - measure flow on last (ie., fastest flowing) hole (bucket and stopwatch) then immediately measure pressure rise on this same fastest flow, flattest angle hole. Measure pressures as per procedure used for other PT hole data collection undertaken on 10-11/6/91 (ref. Appendix A). Once static pressure achieved (overnight say) put pressure gauge on slowest flowing hole, then open fastest flow hole and record flow; measure at 30", 1 min etc., to determine pressure drop on this hole with flow from all other holes going full stream. Leave the area to stay draining - continue to record pressures from low flow hole on same basis as other PT holes.

- Determine overburden vertical gradient data by use of shallow multipoint piezometer installations.

- Consider monitoring pressure head data (pore-pressure) in trench material - low air entry piezometer tips are recommended (ref. Appendix C). A cone headtip driven into the clayey and/or silty sections of trench material low down would

be advantageous. The tips should be VW or other appropriately rugged readout construction.

- A sensitivity check on soil-mechanics failure behaviour of trench material - under pore pressure loadings etc., is needed. This should aid in evaluation of underhand or overhand stoping approaches. These should consider soil mechanics behaviour of trench infill. Long cable bolts could be considered for rock crown with soil anchors in trench material to enhance splitset pullout resistance.

- May need to consider higher pore pressures and lag times in trench as opposed to drainage of evaporites - could be an important consideration for mine hangingwall stability.

- Should consider developing lowest level underground drifts as drainage gallery for drainhole drilling as this will maximize gravity assistance of drainage.

- For holes in evaporites, install long casings on any holes planned for drainage of high flows in these materials. Carefully check cement bonding on a rock bolt pull test to establish minimum bond for the casing, or design casing lengths on 50 psi bond between the casing and the hole.

- Use flowmeter on individual Aardvark holes to monitor "blocking off" behaviour and change in flow patterns between holes when new holes are being drilled or other events are happening.

- For monitoring pressure points installed on 11/6/91, elevation data and trench intersection elevations, along-hole lengths and geology data are required. This type of data is also needed for any new holes.

- Develop understanding of along-strike pressure response behaviour (from various existing and new points) - try to develop comprehension of cross strike characteristics - evaporite, trench, carbonate, quartzite. There may be some variations depending on percentage fines and associated hydraulic conductivity characteristics of trench and margin materials.

- Try casing one long-hole/Aardvark hole right through the trench zone - into karstified zone in evaporites - see if it makes just as much water as a hole screened through the trench - see if it runs clear - if it does, it may be possible to eliminate a lot of stainless steel screen installations.

- Maintain graphical and spreadsheet approach of monitoring, but concentrate on local area by area records of flow (pumping) and PT monitoring pressures. Then integrate into general picture.

- Check correlation of surface sinkholes with locations of old exploration hole collar positions.

3.0 DEWATERING EVALUATION

3.1 Pressure/Flow Testing

In order to provide some guidance on areas where underground dewatering points would be most effective, some simplified transmissivity (and approximate permeability) calculations were undertaken using the pressure/flow test data from the various pressure monitoring points installed during the second site visit. Figure 7 which shows the plot from hole PT-6 in the 803 sump area is characteristic of the results obtained from one such test.

Response characteristics for this hole and for most of the holes tested showed classic influence of recharge, and overall, suggest that transmissivities are mostly in the order of 10^{-6} to 10^{-4} m^2/sec, with derived hydraulic conductivities in the range of 10^{-4} to 10^{-2} cm/sec.

The raw data collected from these flow/pressure tests is presented in Appendix B while the summary results are presented on Table 1. In total, pressures and flows were monitored at 10 of the 12 gauged pressure monitoring locations shown on Figure 6. As indicated on Table 1, in order to evaluate approximate permeabilities, various assumptions were made on the actual widths of permeable material through the trench zone and the karst contact. The results of these calculations, as well as the basic transmissivity data that are summarized on Table 1 are largely based on interpretation of the drillers' logs of material zone intersection. Even though there are wide variations in responses, the order of magnitude results seem fairly consistent. Nevertheless, as considerable uncertainty remains in the assumptions of actual "permeable" widths and in representative "flows" for calculating insitu permeabilities, the numbers given on Table 1 should be considered only as guideline numbers.

The data included in Appendix B, on which the calculations have been based, was collected from simple pressure drawdown and recovery tests. Flows were measured with a bucket and stop-watch and, where possible, downward inclined holes were utilized to avoid problems arising from only partial filling of the hole during the test period. Although one or two locations could be selected close to areas known to be tapped into zones of active, effective dewatering, most of the pressure/flow test holes across the mine site were located into areas of less significant permeability, thus individual hole flows, in some cases, reflect only local effects of drainage of

the rock mass, while in other locations, the aggregate flow from all adjacent drain holes (Aardvark or Longhole) needs consideration in order to reliably define appropriate "flow" magnitude for the calculation of transmissivity and permeability. Nevertheless the "shut-in" pressure head data obtained from the tests does provide a more accurate and wider scale understanding of the development of the drawdown "cone" characteristics than had been available prior to installation of the pressure points; ref. the data listed in the seventh column of Table 1 and summarized on Figure 3 of the Memorandum of July 16th, 1991 included in Appendix A.

Reference to Table 2 indicates that, for the majority of the holes, the actual flows measured are significantly lower than could be sustained by the holes under the known driving head pressures. It is readily apparent, from comparison of the measured initial inflows (shown on the top right side of Table 2) with the flows computed from standard pipe formulas, as included across the bottom of Table 2, that, in nearly all cases, actual flows simply reflect the permeability control of the ground through which the holes are drilled (ref. right side of Table 1). Only PT-3 and PT-7 exhibit flows that are anywhere near full hole flow as would be controlled by "hole diameter". This then reinforces field observations that drainhole drilling needs to be targeted to intersect zones of high intrinsic permeability which can then maximize drainhole effectiveness.

3.2 Aardvark Drainhole Drilling

Over fifty drainholes were placed over the period June to September 1991. The locations of these holes, by area, are shown on Figure 6 and tabulated details for the various holes, by area are given on the following sequence of Tables:

Table No.	Location (ref. Figure 6)		Number of Holes	Estimated September Total Flows (USgpm)
3	A	792 X-Cut	2	225
4	B	804 X-Cut	12	50
5	C	808 X-Cut	10	800
6	D	118-844 Stn	10	1286
7	E	118-3	4	950
8	F	118-1	5	20
9	G	118-853 Stn	7	235
10	H	101-D	2	250

Details of screen length, type and size, and information on encountered geology are also provided on the Tables together with data on initial flow and subsequent flow performance. As is evident from the individual tables, many of the areas have exhibited similar flow behaviour since drilling. This is reflected in the overall drainage effectiveness of the drillholes in these areas. Others have shown marked changes with time (notably the 804 and 118 areas - Tables 4 and 7 to 9). This is also clearly illustrated by the flow and pressure response data for the mine dewatering as graphed on Figures 8 and 9. Some holes have remained more or less consistent since gauging. Others reflect the response behaviour of mine-wide pressures to primary drainhole blockages or to overall flow changes (tailings discharge diversion, heavy rainfall etc.). The flow behaviour, in turn also reflects variations in both elevation and permeability of the trench/karst contact intersection, penetrated by each of the dewatering holes.

Prior to completion of the Aardvark Drill program, the following flow pattern was evident across the mine:

Mine Location	Collection Point	Drainage Areas	Approximate Total Flow (USgpm)
West Area	825 Sump	808 X-Cut 803 X-Cut 804 X-Cut	1000
West Central Area	109 Sump	848 Ramp 851 Decline	350
Central Area	118 Sump	118-1 Area	3200
East Central Area	863 Drift 101D	863 Drift 101 Stopes	150 400
East Area	201 Drift	201 Stopes	200

Total maximum flows of the order of 5300 USgpm were being recorded in mid-June when the above tabulation was compiled. As is evident from comparison of the summary table of individual area flows presented on page 9 the general pattern of underground flows observed in August and September remains much the same as that noted in June, however absolute total flow volumes differ. This is to some extent reflected on the histogram plot given on Figure 8.

Overall, however, the data compiled from the pressure and flow monitoring tests and from the Aardvark drilling intersection records suggests that inflows predominantly reflect proximity to the trench/evaporite contact. The drainhole and pressure head gauge data actually suggests that the groundwater pressure regime across the mine is more or less completely controlled by this high permeability karstic zone adjacent to the trench area. Everything points to it being the main water supply zone, but that water can and often is intersected in trench material, the magnitude of flow intersections being largely governed by the local permeability of the trench material.

3.3 Soil Mechanics Stability Considerations

Other than the issue of the stability of existing areas of stoping which have already extended to the trench margin (Plate 2), two soil mechanics issues are of importance to the effectiveness of dewatering for future mining. These issues relate to control of:

- the stability of hangingwall trench infill soil material and
- the stability of the overburden "seal" over the top of the mine workings.

Although these issues are inextricably linked (as might be indicated by the events leading to muflows and classic inrush behaviour as a result of the drift development into soil filled karst cavity situations - Figure 10), each can be considered separately. The subject of hangingwall stability is addressed in some detail in the memorandum issued on 20th June 1991 contained in Appendix C, while the issue of inflow sensitivity can be evaluated on the basis of published criteria for flow susceptibility of sands and silts.

For sands and sandy silts to "flow", the materials should contain less than 10 to 15 percent fines (in the clay range), (some typical trench material, based on Figure 4, exhibits about 4%). The material should exhibit D_{60} grain size*, between 0.2 mm and 1.0 mm, (the trench material sample has a D_{60} of approximately 0.25 mm) and have uniformity coefficients, U_c between 2 and 5* (the trench material sampled for Figure 4 has a somewhat higher value of 32.9). The sampled trench

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* D_{60} = grain size such that 60% of the particles are smaller than that size.
$U_c = \frac{D_{60}}{D_{10}}$ - where D_{60} is as defined above and D_{10} is the grain size for which 10% of the particles are smaller than that size.

infill material from the 841 stope therefore conforms to two of the above criteria. However, some parts of the mudrush outflow material shown on Plate 2 are of quite uniform gradation and would conform to the uniformity criterion. Thus, if similar material occurs elsewhere within the trench infill and if such materials are not drained, or they become subjected to high water-pressures, these types of materials may "flow" under adverse hydraulic gradients, as indeed has happened before - (ref. Plate 2), the 110 Drift Inflow, 1979; and the 104 Stope Inflow (during mine shut down - 1982 - 89).

Even without mudrush inflows developing into the underground workings from the trench material, surface sinkhole development could continue to occur due simply to excessive fines washout through the dewatering holes. A possible mechanism for this type of sinkhole development process is illustrated on Figure 11. As the main zone of underground flow is likely the karstic hangingwall zone of the evaporites at the margin with the trench, the most likely areas for future sinkhole development would be where the permeable, migration susceptible, lower sand unit directly overlies the karst contact zone or directly overlies permeable bedrock hydraulically connected to the main karst conduit zones.

This interrelationship is addressed in more detail in Golder Associates report 911-1726-1 on the overburden hydrogeological aspects of the property.

4.0 SUMMARY CONCLUSIONS

4.1 Immediate Practical Considerations

As outlined in Section 3 and in the various memoranda contained in Appendices A and C, several technical issues of a practical nature directed to improving dewatering performance, have been addressed. In particular, effective screening approaches have been implemented on many of the over 50 new dewatering holes, and on the fully screened replacement holes drilled into the 118 sump area to take the dewatering load off the critical, but open and blockage-prone, long-holes shown on Plate 1.

After some initial teething troubles during the first few trial screen insertions into the Aardvark drainholes, installation of wire-wound, 50 slot stainless screen became a routine operation. Although some technical concerns remain with respect to ensuring complete sealing of the annulus around the screens in order to prevent any flow by-passing the screen to the collar, most of the installations seem to have, so far, functioned successfully. Only in a few instances has installed screen blocked off completely or been by-passed by flow volumes (due to flow down and through the borehole annulus to the hole collar). Only by regular screen cleaning (either by surging or flushing - possibly with an air lance) will it be possible to minimize potential blockage problems. Various possible measures to alleviate by-pass flow problems (particularly in holes where ongoing continual solution of evaporitic borehole walls could continue) are outlined in Appendix A - ref. Figure 1 to the Memorandum dated July 16th, 1991.

4.2 Other Technical Issues

When all the data collected from the underground dewatering program presented graphically in the figures accompanying this report, are reviewed in the light of the information obtained from the surface drilling and mapping information (ref. also GA Report 911-1726-1) the major conclusions relating to achieving effective dewatering of the karstic hangingwall area of the trench zone can be listed as follows:

(i) Underground Aspects

- Screened holes seem to be functioning satisfactorarily

- Holes targeted into zones of high transmissivity/high head function best
- Locations and numbers of drainholes can be optimized to suit variations in actual insitu hydraulic conductivities

(ii) Soil/Overburden Aspects

- water flow to the mine workings seems to preferentially occur vertically downward through the overburden soils and then laterally via the weathered rock surface, the karst zone and then the trench margins

- three main permeable conduits of flow to the underground workings likely exist:
  - along the trench, in the karst evaporites (from the ends)
  - directly downwards from the river/shallow overburden system
  - from the lower sand sequence in the MeadowBrook Valley area

- the variability in infill materials permeability within the gypsum karst voids close to the trench exert significant control on flow behaviour into the underground workings

- of the major sub units recognized in the overburden stratigraphy, (ref. report 911-1726-1) of most importance is the lower (water bearing) sand horizon which extends northwards from the mine beneath the till under the Meadowbrook Valley

- observations suggest that the river alluvium/upper sand is likely in direct contact with permeable gypsum bedrock close to or beneath the river to the north-west of the 804-808 area and also in the region north of the 118 sump. This is the area of the river that dried up following diversion of the discharge flows from the tailings area. (This suggests that approx. 800-1000 USgpm enters the mine from the river in this area - ie. approx. 1/3 of total mine discharge, at periods of extreme low river flow).

- a probable method of development of the more recent sinkholes has been formulated - (ref. Figure 11). Several seem to have preferentially developed in areas where the gypsum bedrock is covered by less than 20 m. of overburden (ref. GA report 911-1726-1).

(iii) Groundwater Levels

- Pressure head data from underground monitoring holes and data from surface piezometer monitoring indicates that the geometry of the *drawdown cone* is complex and locally very spiky. Significant drawdown extends both east and west along the contact margin. Drawdown also extends northwards along the MeadowBrook valley, (ref. GA report 911-1726-1) but very low pressure conditions only exist locally to the dewatering hole positions (ref. Tables 2 and 11 and Figures 8 and 9).

- Pressure heads seem to respond directly to additional drainholes from underground; blockages of holes in the 118 sump are reflected in changed pressure and flows on other drain locations - the system is fully interconnected (ref. Figures 8 and 9).

5.0 RECOMMENDATIONS

5.1 Underground Aspects

Based on the performance of recently installed drainholes, dewatering control at Gays River can be achieved by careful positioning of fully screened holes. Matching of screen to encountered fine materials within the holes is however *essential* to avoid piping fines out of the formation materials, but this works against maximization of flows from individual holes. Thus, as large as feasible a hole size is required in order to achieve required volumes of flow. Measures to control migration of fines if the problem becomes severe are outlined on Figure 1 to the Memorandum dated July 14th, 1991 - ref. Appendix A, while measures to control potential solutioning of dewatering holes, collared and/or drilled through, from or into the evaporite near the trench margin area are provided on Figure 1 of the Memorandum dated July 16th, 1991 - ref. Appendix A.

Despite the general success of the screened drainhole installations in preventing flow blockages or influx of significant quantities of material (gravels principally), some technical concerns still exist relating to:

- maintaining "stability" of installed drainholes due to fines migration, and
- controlling soil stability of the hangingwall during and prior to stope filling, (ref. provisions outlined in Appendix C)

From the underground perspective, with care, it is considered that dewatering control can be maintained of the workings, such that they can be maintained in a depressurized state. Such control may only be feasible, however, provided that comprehensive daily monitoring is maintained not only of mine-wide flows and pressures but also of local stope area pressures as suggested in Appendix C. In this regard, consideration should be given to installing permanent pressure transducer monitoring points in critical areas, possibly hooked up to a surface control system, such that when required, drainage holes can be flushed to clean the screens, on the one hand, or larger holes with finer slot size screens can be installed or mitigation measures implemented to control blockages or excessive fines washout.

The issue of fines washout should also be addressed more quantitatively. Simple hydrometer checks, on a regular basis, of water emanating from the various individual drainholes would help establish trends in percentage fines washout. Bulk estimates, with time, of fines collecting in the sumps would also aid understanding. Although it is anticipated that a graded "gravel pack" type of filter will eventually develop around each of the Aardvark Drainhole screens, this may take some time. It is important, therefore, that conditions should continue to be monitored in order that undue volumes of fines washing out of the trench zone and karst voids can be detected and control measures implemented, if required.

Remedial solutions may also need to be implemented when or if serious short-circuiting of flow occurs around any screen annulus directly to the collar, thus by-passing fines around the screen. Long term solutioning problems developing within the holes into or through the evaporite units also needs to be carefully monitored and remediation measures taken, if and when required. Of particular note in this regard, is hole No 13 which was collared and drilled through evaporites from the 118-844 Drill Station. This hole initially exhibited a flow of only 15 gpm, however, by the end of October, 1991, this hole alone was delivering 1200 USgpm (ref. Table 6).

5.2 Other Issues

With regard to surface infiltration from the River, because of the supposed interconnectivity of the flow systems vertically, as illustrated on Figures 10 and 11, it is particularly important that future inundation of the river flood plain be prevented (ie., including spring floods)

Specific options to achieve this could involve:

(i) provision of an upstream control structure,
(ii) channelization of the river
- straighten and deepen existing course, or
- place river in clay-lined cut (especially on the west side of the mine area), or
- run stream through culvert, or

(iii) completely divert the river elsewhere (ie. around the mine site)

In addition to the current underground program of drainholes, to assist in developing full dewatering of the mining zone, consideration should also be seriously given to introducing some "*end zone*" surface dewatering wells to take the load off the underground system. Further, there may be some merit to considering installing surface dewatering wells into zones of permeable gypsum beneath the overburden in order to intercept river infiltration and provide subdrainage of overburden water before it enters the deeper karst zone. This approach may be more effective than either a) installing numerous additional underground drainholes for providing subdrainage or b) directly installing overburden dewatering wells *per se* to dewater the lower sand unit.

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TGC/se

TABLE 1

HYDROGEOLOGICAL CHARACTERISTICS OF FLOW ZONES AT PRESSURE MONITORING HOLES

FIELD INFORMATION							DRAINAGE TEST RESULTS					LUGEON CALCS.	
TEST LOCATION	HOLE NO.	TRENCH INTERSECTION DEPTHS START	FINISH	THICKNESS m	FLOW l/min MAX MIN	HEAD m MAX MIN	TRANSMISSIVITY T m^2/s	STORATIVITY S	SPECIFIC STORAGE Ss m^-1	COEF. OF CONSOL. m^2/s	HYDRAULIC CONDUCTIVITY k cm/s	Lu	k cm/s
PT-1	U11840	17.00	23.00	5.00	25.00	43.80	3.40E-05	5.21E-06	1.04E-06	6.53E+00	6.81E-04	11	1.14E-04
				5.00	15.60	42.00	6.70E-06	1.93E-04	3.86E-05	3.47E-02	1.34E-04	7	7.43E-05
PT-2	U639	46.60	49.09	2.49	44.00	10.50	2.60E-05	1.79E-03	7.19E-04	1.45E-02	1.04E-03	168	1.68E-03
				2.49	3.90	1.40	2.90E-06	2.00E-04	8.03E-05	1.45E-02	1.16E-04	112	1.12E-03
PT-3	LONGHOLE	15.20	17.70	2.50	60.00	14.00	8.17E-05	2.11E-05	8.45E-06	3.87E+00	3.27E-03	171	1.71E-03
				2.50	57.00	9.80	7.76E-05	2.01E-05	8.03E-06	3.87E+00	3.10E-03	233	2.33E-03
PT-4	U597	20.95	23.47	2.52	7.50	4.90	1.02E-05	8.34E-07	3.31E-07	1.22E+01	4.05E-04	61	6.07E-04
				2.52	5.50	1.40	8.56E-06	6.07E-05	2.41E-05	1.41E-01	3.40E-04	156	1.56E-03
PT-5	U641	38.85	40.54	1.69	75.00	12.60	3.60E-05	6.24E-04	3.69E-04	5.77E-02	2.13E-03	352	3.52E-03
				1.69	8.80	2.10	4.20E-06	7.28E-05	4.31E-05	5.77E-02	2.49E-04	248	2.48E-03
PT-6 *	U649	36.90	38.10	1.20	567.75	7.73	9.47E-04	8.81E-06	7.34E-06	1.08E+02	7.90E-02	6117	6.12E-02
				1.20	52.00	4.20	5.51E-05	5.12E-07	4.27E-07	1.08E+02	4.59E-03	1032	1.03E-02
PT-7	LONGHOLE	30.00	32.00	2.00	44.40	5.20	4.40E-05	9.67E-07	4.83E-07	4.55E+01	2.20E-03	427	4.27E-03
				2.00	44.40	1.40	4.40E-05	9.66E-07	4.83E-07	4.55E+01	2.20E-03	1586	1.59E-02
PT-8	D11801	30.00	33.00	3.00	2.20	21.09	**	**	**	**	**	**	**
				3.00	2.20	2.00	**	**	**	**	**	**	**
PT-9	U563	42.07	42.99	0.92	104.00	11.90	5.00E-05	3.19E-03	3.47E-03	1.57E-02	5.43E-03	950	9.50E-03
				0.92	17.90	1.40	8.67E-06	5.53E-04	6.01E-04	1.57E-02	9.42E-04	1390	1.39E-02
PT-10	U118-26	15.00	20.00	5.00	50.00	37.10	2.20E-04	2.38E-04	4.76E-05	9.24E-01	4.40E-03	27	2.70E-04
				5.00	50.00	35.70	2.18E-04	1.47E-05	2.94E-06	1.48E+01	4.36E-03	28	2.80E-04
RANGE OF RESULTS				5.00	567.75	43.80	9.47E-04	3.19E-03	3.47E-03	1.08E+02	7.90E-02	6117	6.12E-02
				0.92	3.90	1.40	2.90E-06	5.12E-07	3.31E-07	1.45E-02	1.16E-04	7	7.43E-05
AVERAGE				2.63	61.86	13.51	3.36E-05	3.58E-05	1.59E-05	9.40E-01	1.50E-03	681	1.98E-03

* Maximum Flow Value assumes flow for entire 803 Sump (150 USgpm)

** Insufficient flow volume to permit rational testing

NOTE: The results of the k calculations are very sensitive to the "thickness" of assumed permeable ground.

TABLE 2

COMPARISON OF FLOW DATA MEASURED FROM PRESSURE TEST HOLES WITH STANDARD PIPE-FLOW CALCULATIONS FOR SAME DRIVING HEAD

TEST HOLE PRESSURE & FLOW DATA

TEST SITE	HOLE No.	MEASURED PRESSURE (psi)	EQUIV. HEAD (m.)	COLLAR ELEV'N. (m.)	TOTAL H20 ELEV. (m.)	HOLE INCL. (Deg.)	HOLE LENGTH (m.) @ INTERS'N OF H2O	RISE (m.)	INTERS'N ELEV'N (m.)	H2O HEAD @ INTRS'N (m.)	HOLE DIAMETER (inches)	HOLE DIAMETER (metres)	MEASURED FLOWS V m/s	MEASURED FLOWS Q L/min	MEASURED FLOWS USGPM	MEASURED FLOWS IGPM
PT-1	U11840	62.20	43.80	410.00	453.80	15.00	17.00	4.40	414.40	39.40	2.50	0.0635	0.13	25.00	6.61	5.50
PT-2	U639	14.83	10.50	441.04	451.54	2.00	42.13	1.47	442.51	9.03	2.50	0.0635	0.23	44.00	11.62	9.68
PT-3	Longhole	19.91	14.00	421.50	435.50	1.00	15.20	0.27	421.77	13.73	1.88	0.0476	0.56	80.00	15.85	13.20
PT-4	U667	6.97	4.90	441.88	446.78	1.00	20.95	0.37	442.23	4.53	2.50	0.0635	0.04	7.50	1.98	1.65
PT-5	U641	17.92	12.60	428.66	441.26	15.00	38.85	10.08	438.74	2.54	2.50	0.0635	0.30	75.00	19.82	16.50
PT-6	U649	10.95	7.70	427.35	435.05	2.00	35.60	1.29	428.64	6.41	2.50	0.0635	0.27	52.00	13.74	11.44
PT-7	Longhole	7.40	5.20	412.00	417.20	5.00	30.00	2.61	414.61	2.59	1.88	0.0476	0.42	44.40	11.73	9.77
PT-8	D11801	30.00	21.09	428.00	449.09	20.00	105.00	35.91	463.91	-14.82	2.50	0.0635	0.01	2.20	0.58	0.48
PT-9	U663	16.93	11.90	471.81	483.71	24.00	42.07	17.11	488.92	-5.21	2.50	0.0635	0.55	104.00	27.48	22.88
PT-10	U118-26	52.77	37.10	407.50	444.60	15.00	15.00	3.88	411.38	33.22	1.88	0.0476	0.47	60.00	13.21	11.00
PT-11	804	40.00	28.12	416.00	444.12	1.00	13.10	0.23	416.23	27.90	1.88	0.0476	0.18	20.00	5.28	4.40
PT-12	64-R1-H	34.00	23.91	417.50	441.41	18.00	6.70	2.07	419.57	21.83	4.00	0.1016	0.16	80.00	21.14	17.60

EMPIRICAL AND SEMI-EMPRICAL RESULTS

(PIPE FLOW EQUATIONS)

TEST HOLE	BERNOUILLI VALUES V m/sec	BERNOUILLI VALUES Q USGPM	BERNOUILLI VALUES Q IGPM	HAZEN WILLIAMS V m/sec	HAZEN WILLIAMS Q USGPM	HAZEN WILLIAMS Q IGPM	MANNING APPROACH V m/sec	MANNING APPROACH Q USGPM	MANNING APPROACH Q IGPM	SWAMEE & JAIN EQUATION V m/sec	SWAMEE & JAIN EQUATION Q USGPM	SWAMEE & JAIN EQUATION Q IGPM	MASSEY MODIFIED COLEBROOK-WHITE APPROACH V m/sec	MASSEY MODIFIED COLEBROOK-WHITE APPROACH Q USGPM	MASSEY MODIFIED COLEBROOK-WHITE APPROACH Q IGPM
PT-1	6.13	407.72	339.47	6.2006	411.49	342.61	7.0409	353.29	294.15	6.7195	437.62	364.28	7.2704	364.80	303.74
PT-2	2.83	146.67	122.29	2.6677	130.35	108.53	2.4285	121.86	101.48	2.9568	148.48	123.61	2.7922	140.10	116.66
PT-3	2.64	74.60	62.11	2.3314	66.80	54.79	2.1522	60.75	50.58	2.6832	75.73	63.05	2.6356	74.39	61.94
PT-4	2.69	136.22	112.58	2.4854	124.71	103.83	2.3312	116.97	97.39	2.8373	142.37	118.53	2.6883	134.89	112.31
PT-5	3.41	170.83	142.31	3.0811	154.60	128.72	2.8443	142.72	118.83	3.4775	174.49	145.28	3.2302	162.08	134.96
PT-6	2.74	137.38	114.38	2.4420	122.53	102.02	2.2936	115.08	95.81	2.7903	140.01	116.57	2.6479	132.86	110.62
PT-7	1.21	34.25	28.51	0.9922	28.01	23.32	0.9758	27.54	22.93	1.2138	34.25	28.52	1.2149	34.29	28.55
PT-8	2.77	138.75	115.53	2.3738	119.11	99.17	2.2342	112.10	93.34	2.7163	136.30	113.48	2.5842	129.67	107.96
PT-9	3.22	161.80	134.71	2.8948	145.25	120.94	2.6847	134.71	112.16	3.2783	164.50	136.96	3.0831	153.70	127.97
PT-10	4.06	114.61	95.43	3.8944	104.27	86.82	3.2962	93.03	77.46	4.1124	116.07	96.64	3.9754	112.20	93.42
PT-11	6.23	176.80	147.21	5.9472	168.75	140.50	5.3547	151.94	126.51	6.5676	186.92	155.63	5.3065	150.63	125.41
PT-12	8.30	1086.60	887.22	9.7364	1250.92	1041.52	7.9381	1019.67	848.96	9.8707	1267.91	1055.66	8.9502	1149.67	957.22

TABLE 3

AARDVARK DRAINHOLE SUMMARY, 792 X-CUT (AREA-A ON FIGURE 6)

Hole Number	Dip (deg)	First Trench metres	First Gypsum metres	Second Trench metres	Second Gypsum metres	Third Trench metres	Third Gypsum metres	Total Hole Length (m)	Screen Length (m)	Estimated Water Flow (USgpm)
1	+10	12.2	15.2	19.2	20.1	22.6	27.1	31.1	6.1 *	none
2	+25	12.8	14.9	18.9	19.5	31.7	33.5	37.8	3.0 & 12.2	225
Total								68.9		225

NOTES: * Lost first drop–off bit at 15 m into hole
1 Both holes drilled at 4.25" diameter and screened with 3" telescopic stainless steel screen (2.625" dia.). 50 Slot Screen Size.
2 Data to September 30, 1991

TABLE 4

AARDVARK DRAINHOLE SUMMARY, 804 X-CUT (AREA-B ON FIGURE 6)

Drill Order Number	Ring Number	Hole Number	Dip (deg)	First Trench metres	First Gypsum metres	Second Trench metres	Second Gypsum metres	Total Hole Length (m)	Screen Length (m)	Length of blank after screen (m)
1	1	1	0	13.1	15.5			21.0	6.1	
2	1	2	+6	9.1	13.7			19.5	9.1	3.0
3	1	3	+12	8.2	12.8			17.4	6.1	3.0
4	1	4	+18	8.7	13.7	16.2	16.8	18.9	9.1	3.0
5	1	5	+24	8.7	13.7			22.9	9.1	6.1
6	2	5	+24	11.0	13.7	16.5	17.4	20.7	9.1	
7	2	4	+18	12.2	17.7	21.9	25.0	28.3	12.2	3.0
8	2	3	+12	12.8	17.4	18.9	21.3	24.7	9.1	3.0
9	2	2	+6	13.7	15.8			20.4	6.1	3.0
10	2	1	+6	18.3	19.5			24.1	6.1	
11	3	5	+24	10.7	12.2			16.8	6.1	
12	3	4	+18	11.3	15.2			21.3	9.1	
Total								256.0		

NOTES: 1 All holes screened with 3" telescopic, stainless steel screen. Slot size – 50 Slot (0.050")
2 Peak flow from 804 X–cut Holes – 300 USgpm in July 1991. Flow from 804 X–cut Holes in October 1991 – 50 USgpm.
3 Data to July 13, 1991

TABLE 5

AARDVARK DRAINHOLE SUMMARY, 808 X-CUT (AREA-C ON FIGURE 6)

Drill Order Number	Ring Number	Hole Number	Dip (deg)	First Trench metres	First Water metres	First Gypsum metres	Total Hole Length (m)	Screen Length (m)	Screen Type
1	3	1	0	11.9	12.2	18.2	18.2	9.1	PVC
2	2	1	0	12.8	18.2	19.8	19.8	12.2	PVC
3	1	1	0	15.8	17.1	20.4	30.5	12.2	Steel
4	3	2	+6	9.8	10.1	10.7	17.4	9.1	Steel
5	3	3	+12	7.9	8.5	11.3	14.3	6.1	Steel
6	3	4	+18	6.7	7.3	9.4	11.3	6.1	Steel
7	3	5	+24	5.5	5.8	7.3	9.9	6.1	Steel
8	4	1	0	10.1	10.4	17.1	20.7	9.1	Steel
9	4	2	+6	8.8	9.4	14.3	17.1	9.1	Steel
10	2	2	+6	11.6	11.9	20.7	23.8	12.2	Steel
Total							180.9		

NOTES: 1 PVC Screen is 1.5", 10 slot size.
2 Steel Screen is 2.625" o. d., 50 slot size stainless steel screen.
3 Total water flow from 808 X-cut is estimated at 800 USgpm, and has maintained this flow since July 1991.
4 Data to June 30, 1991

TABLE 6

AARDVARK DRAINHOLE SUMMARY, 118-844 DRILL STATION (AREA-D ON FIGURE 6)

Drill Order Number	Hole Number	Dip (deg)	First Trench (metres)	First Carbonate (metres)	First Quartzite (metres)	Total Hole Length (m)	Screen Length (m)	Estimated Water Flow After Drilling (USgpm)	Estimated Water Flow Oct 17 (USgpm)
Holes Drilled in July									
1	11	+17	44.2	44.5	45.1	46.3	—	15	15
2	3	+17	—	31.7	32.9	33.8	—	dry	dry
3	9	+17	—	37.5	38.1	43.3	—	5	5
4	10	+21	—	45.7	47.5	50.9	—	5	5
5	1	+17	—	28.0	28.7	35.1	—	1	1
6	6	+21	—	37.8	41.8	44.8	—	dry	dry
7	7	+17	—	31.1	32.6	35.7	3.0 *	60	60
8	8	+21	—	45.4	46.0	50.9		dry	dry
Holes Drilled in August									
9	12	+27	—	54.3	—	54.9		dry	dry
10	13	+28	46.6	55.5	—	58.5	9.1 **	15	1200
Total						454.2		101	1288

NOTES: * 4.0" dia. (4.5" o.d) screen
** 2.625" o. d., 50 slot size, stainless steel screen (3" telescopic).
1 Data to August 31, 1991

TABLE 7

AARDVARK DRAINHOLE SUMMARY, 118-3 AREA (AREA-E ON FIGURE 6)

Ring Number	Hole Number	Dip (deg)	First Gypsum metres	First Trench metres	Second Rock metres	Second Trench metres	Third Rock metres	Total Hole Length (m)	Screen Length (m)	Estimated Water Flow After Drilling (USgpm)	Estimated Water Flow Oct 17/91 (USgpm)
Holes Completed in July											
1	2	+27.5	11.3	28.7	37.2	38.1	42.7	44.8	15.2	250	800
1	1	+23	18.3	41.8	58.8	—	—	61.0	18.3	350	100
Holes Completed in August											
2	2	+27.5	11.3	29.0	36.3	38.7	47.5	51.8	12.2	600	200
2	1	+23	16.2	26.5	28.3	—	—	35.1	6.1	100	50
Total								192.7		1300	950

NOTES: 1 Holes screened with 4" stainless steel screen (4.5" o.d.)
Slot size – 50 slot (0.050")
2 Data to August 31, 1991

TABLE 8

AARDVARK DRAINHOLE SUMMARY, 118-1 AREA (AREA-F ON FIGURE 6)

Hole Number	Dip (deg)	First Trench metres	First Gypsum metres	Total Hole Length (m)	Screen Length (m)	Estimated Water Flow (USgpm)	Details
1	+26	—	—	16.8	—	—	Hole Ended in quartzite at lost drill rod.
2	+26	19.5	21.3	25.3	3.0	—	3" telescopic screen Hole ran 200 USgpm initially but stopped when 118–3 holes surged.
3	+26	18.0	18.2	20.4	3.0	5	3" telescopic screen
4	+25	18.9	19.5	22.3	3.0	15	4.5" o. d. screen
5	+28	16.5	17.7	20.7	6.1	1	4.5" o. d. screen
Total				105.5		21	

NOTES: 1 All wells used 50 Slot Screen Size
2 Data to August 30, 1991

TABLE 9

AARDVARK DRAINHOLE SUMMARY, 118-853 DRILL STATION (AREA-G ON FIGURE 6)

Drill Order Number	Hole Numbe	Dip (deg)	First Carbonate metres	First Trench metres	First Gypsum metres	Second Carbonate metres	Second Trench metres	Second Gypsum metres	Total Hole Length (m)	Screen Length (m)	Initial Water Flow (USgpm)	Water Flow Oct 17 (USgpm)
1	2	+16	16.8	21.6	26.8				31.1	12.2	15	5
2	1	+11	22.9	23.8	28.3				31.1	12.2	60	30
3	3	+11	16.8	21.3	27.1				30.5	12.2	1	5
4	4	+16	20.1	21.3	26.8				29.6	12.2	1	1
5	5	+11	23.5	24.1	30.2				38.7	6.1	1	30
6	7	+5	23.8	26.2	29.6				38.7	12.2	60	15
7	6	+5	23.2	25.9	28.7	33.8	34.4	35.7	37.8	12.2	250	150
Total									237.5		388	236

NOTES: 1 Telescopic 3" (2.625" o. d.) stainless steel 50 slot size screen.
2 Holes collared in quartzite.
3 Data to October 17, 1991

TABLE 10

AARDVARK DRAINHOLE SUMMARY, 101 AREA (AREA-H ON FIGURE 6)

Hole Number	Dip (deg)	First Trench metres	First Gypsum metres	Total Hole Length (m)	Screen Length (m)	Estimated Water Flow (USgpm)
1	+25	11.6	32.9	39.0	21.3	50
2	+25	8.5	32.6	36.3	24.4	200
Total				75.3		250

NOTES: 1 Both holes drilled at 4.25" diameter and screened with 3" telescopic stainless steel screen (2.625" dia.). 50 Slot Screen Size.
2 Data to September 30, 1991

November, 1991

911-1721

TABLE 11
PRESSURES AND HEAD ELEVATIONS
UNDERGROUND MONITORING LOCATIONS - 1991

POINT	LOCATION	GAUGE ELEV. m *	PRESSURES (psi) 11-Jun	19-Jun	27-Jun	03-Jul	10-Jul	18-Jul	26-Jul	31-Jul	09-Aug	15-Aug	22-Aug	05-Sep	12-Sep	18-Sep	26-Sep	04-Oct	09-Oct	17-Oct
PT-1	118 Sump	410.00	62.0	30.5	26.0	31.0	41.0	50.0	36.0	30.0	23.0	29.0	28.0	40.0	42.0	40.0	47.0	56.0	43.0	31.0
PT-2	825 Ramp	441.04	15.0	14.0	13.0	13.0	13.0	14.0	13.0	10.0	4.0	10.0	12.0	16.0	15.0	15.0	16.0	16.0	17.0	16.5
PT-3	808 Sump	421.50	20.0	6.0	8.0	7.0	9.0	9.0	9.0	7.0	6.0	8.0	8.0	9.0	9.0	9.0	9.0	11.0	11.0	10.5
PT-4	848 Ramp	441.86	7.0	0.0	0.0	0.0	3.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
PT-5	851 Decline	428.68	18.0	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	17.0	16.5	16.5	16.5	16.5	16.5
PT-6	803 Sump	427.35	11.0	8.0	15.0	17.0	21.0	20.0	20.5	23.0	23.0	25.0	26.0	18.0	23.0	21.0	22.0	23.0	22.0	24.0
PT-7	101-Drift	412.00	7.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	0.0	0.0	0.0	0.0	3.5	3.5	3.5
PT-8	119 Sump	428.00	**	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	29.0
PT-9	201 Drift	471.80	17.0	20.0	20.0	20.0	20.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	19.0	19.0	19.0	19.0	19.0	19.0
PT-10	118 Holes	407.50	53.0	29.0	26.0	25.0	32.0	16.0	31.0	25.0	9.0	9.0	9.0	gauges	removed	**	**	**	**	**
PT-12	804 Hole	417.50	**	**	**	**	**	34.0	37.0	40.0	39.0	41.0	39.0	25.0	30.0	27.0	28.0	26.0	24.0	26.0

POINT	LOCATION	GAUGE ELEV. m *	ELEVATION HEADS (m) * 11-Jun	19-Jun	27-Jun	03-Jul	10-Jul	18-Jul	26-Jul	31-Jul	09-Aug	15-Aug	22-Aug	05-Sep	12-Sep	18-Sep	26-Sep	04-Oct	09-Oct	17-Oct
PT-1	118 Sump	410.00	453.7	431.5	428.3	431.8	438.9	445.2	435.3	431.1	426.2	430.4	429.7	438.2	439.6	438.2	443.1	449.4	440.3	431.8
PT-2	825 Ramp	441.04	451.6	450.9	450.2	450.2	450.2	450.9	450.2	448.1	443.9	448.1	449.5	452.3	451.6	451.6	452.3	452.3	453.0	452.7
PT-3	808 Sump	421.50	435.6	425.7	427.1	426.4	427.8	427.8	427.8	426.4	425.7	427.1	427.1	427.8	427.8	427.8	427.8	429.2	429.2	428.9
PT-4	848 Ramp	441.86	446.8	0.0	0.0	0.0	444.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
PT-5	851 Decline	428.68	441.4	440.3	440.3	440.3	440.3	440.3	440.3	440.3	440.3	440.3	440.3	440.3	440.6	440.3	440.3	440.3	440.3	440.3
PT-6	803 Sump	427.35	435.1	433.0	437.9	439.3	442.1	441.4	441.8	443.5	443.5	445.0	445.7	440.0	443.5	442.1	442.8	443.5	442.8	444.2
PT-7	101-Drift	412.00	416.9	415.5	415.5	415.5	415.5	415.5	415.5	415.5	415.5	415.5	415.5	**	**	**	**	414.5	414.5	414.5
PT-8	119 Sump	428.00	**	449.1	449.1	449.1	449.1	449.1	449.1	449.1	449.1	449.1	449.1	449.1	449.1	449.1	449.1	449.1	449.1	448.4
PT-9	201 Drift	471.80	483.8	485.9	485.9	485.9	485.9	484.5	484.5	484.5	484.5	484.5	484.5	484.5	485.2	485.2	485.2	485.2	485.2	485.2
PT-10	118 Holes	407.50	444.8	427.9	425.8	425.1	430.0	418.8	429.3	425.1	413.8	413.8	413.8	**	**	**	**	**	**	**
PT-12	804 Hole	417.50	**	**	**	**	**	441.4	443.6	445.7	445.0	446.4	445.0	435.1	438.6	436.5	437.2	435.8	434.4	435.8

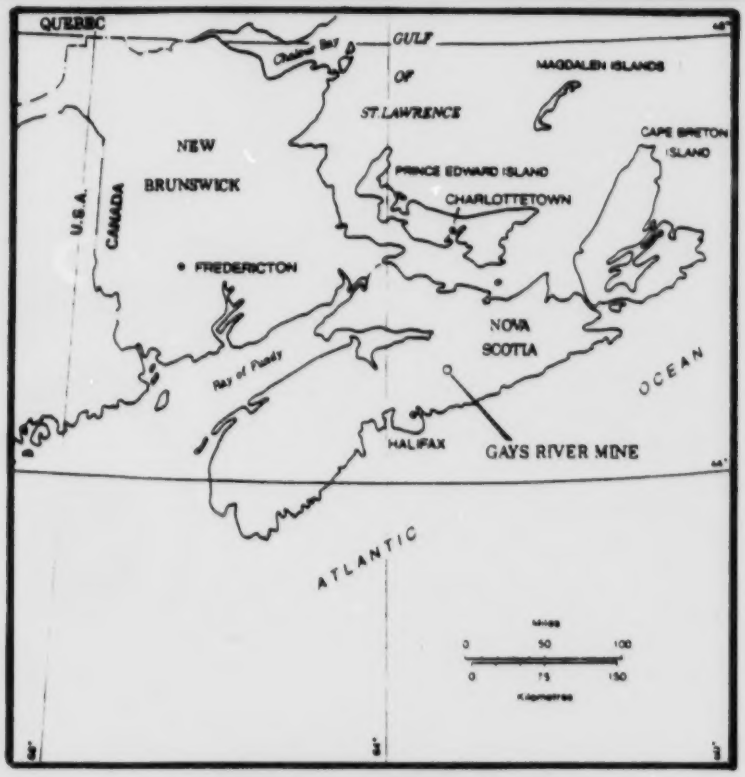
NOTE: * - Elevation to mine datum

** - No data available or zero pressure

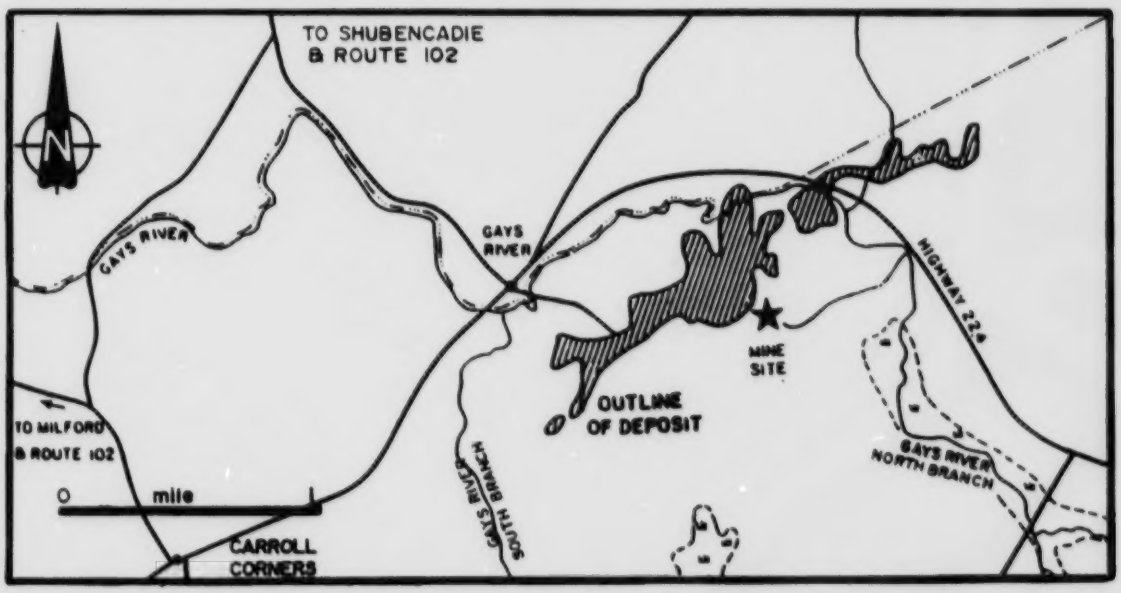
GTD402 WK1

SITE LOCATION PLAN

FIGURE 1



GENERAL LOCATION PLAN



MINE SITE AREA SHOWING GENERAL ARRANGEMENT

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Project 911-1721

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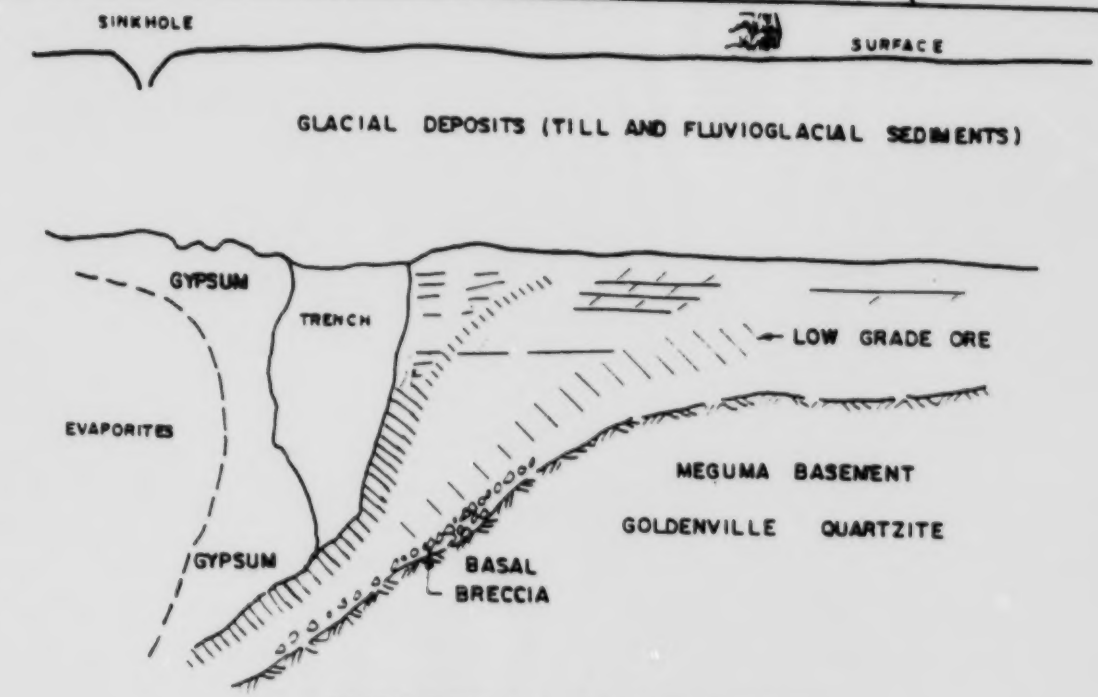
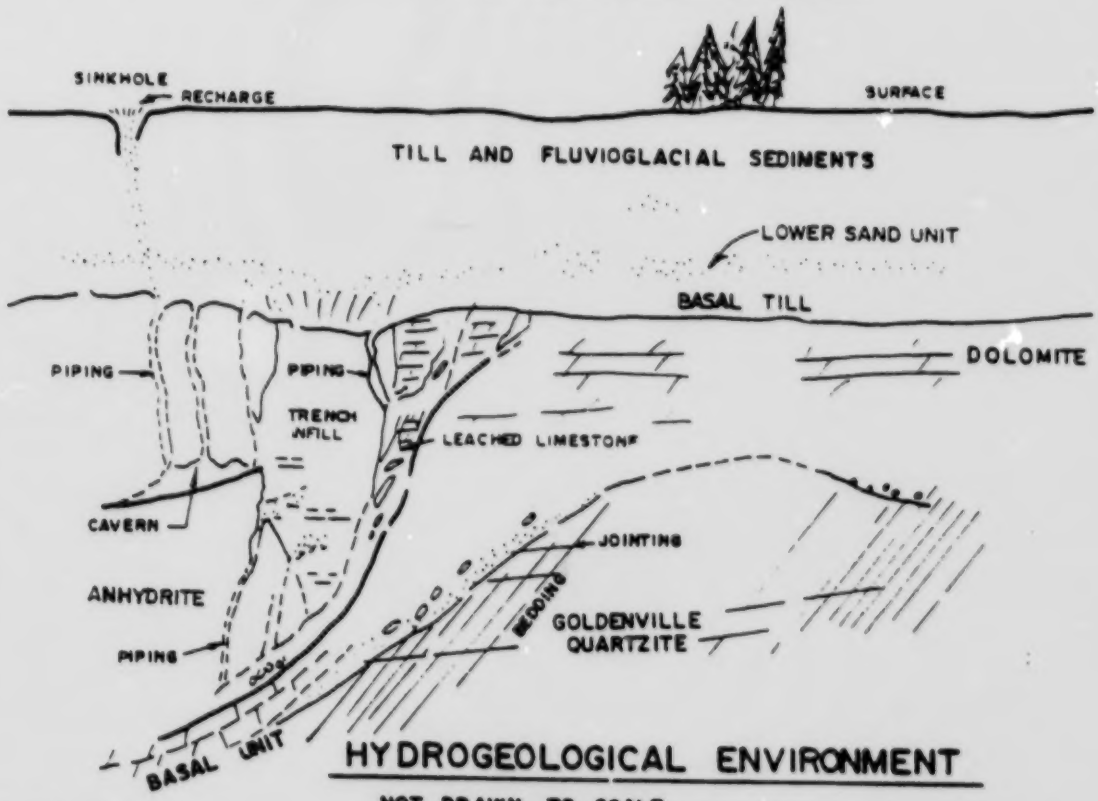
Chkd. TGL

SECTIONS SHOWING GEOLOGICAL AND HYDROGEOLOGICAL ENVIRONMENT OF OREBODY
(MODIFIED FROM SKETCHES BY HANNON + McKEE, 1982)

FIGURE 2



GEOLOGICAL ENVIRONMENT



HYDROGEOLOGICAL ENVIRONMENT

NOT DRAWN TO SCALE

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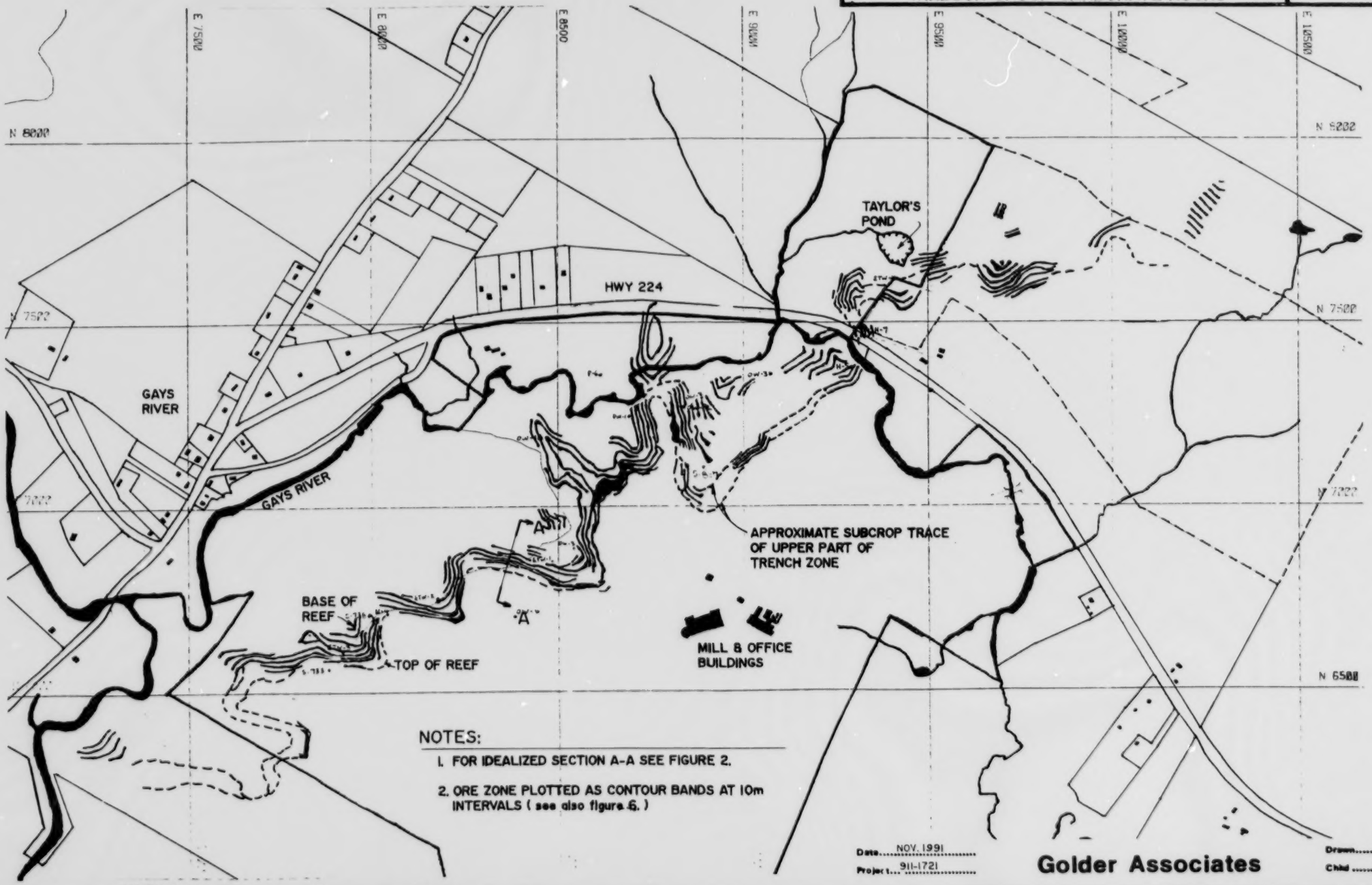
PLAN OF MINE AREA SHOWING COMPLEX GEOMETRY OF ORE ZONE FOLLOWING FOREFRONT REEF FACE OF WINDSOR GROUP CARBONATE ROCKS

FIGURE 3



NOTES:

1. FOR IDEALIZED SECTION A-A SEE FIGURE 2.
2. ORE ZONE PLOTTED AS CONTOUR BANDS AT 10m INTERVALS (see also figure 6.)

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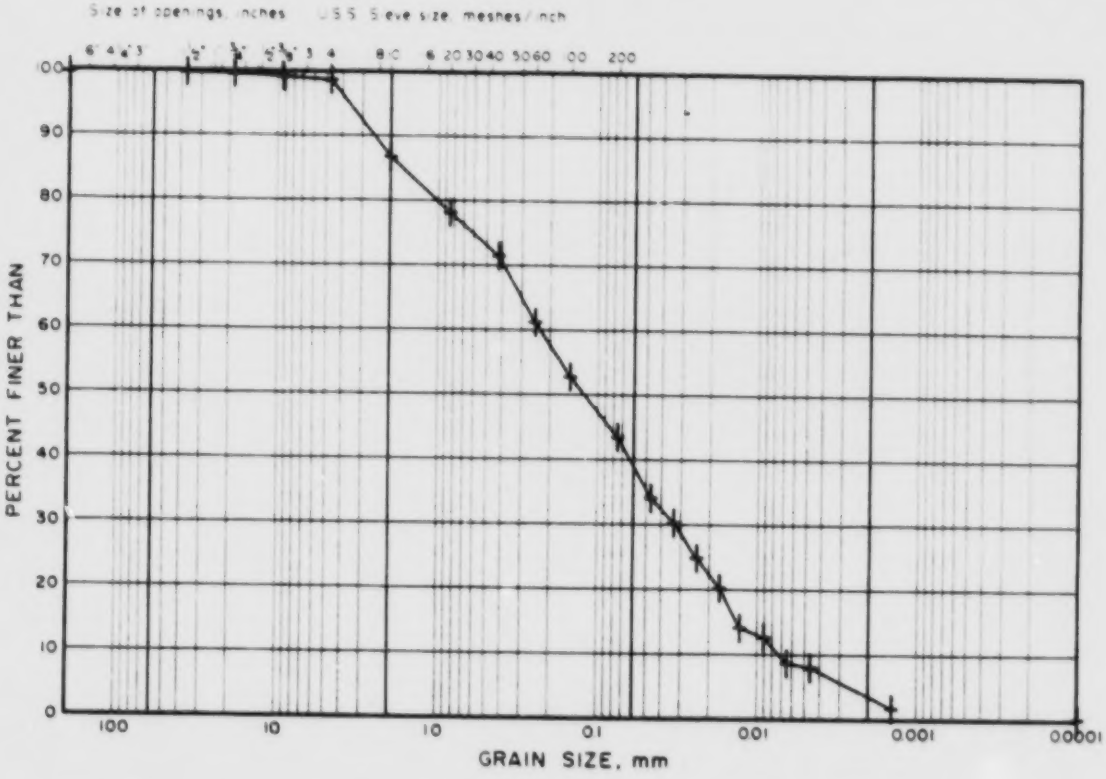
Chkd TAA

GRAIN SIZE DISTRIBUTION AND MATERIAL DESCRIPTION FOR TRENCH SAMPLE FROM 841 WEST STOPE

FIGURE 4



Well graded silt and sand with some gravel
Optimum dry density 126 lbs/ft^3
Optimum moisture content 4%

Liquid limit 22.8%
Plastic limit 20.6%
Plastic index 2.2%
] - for fine fraction

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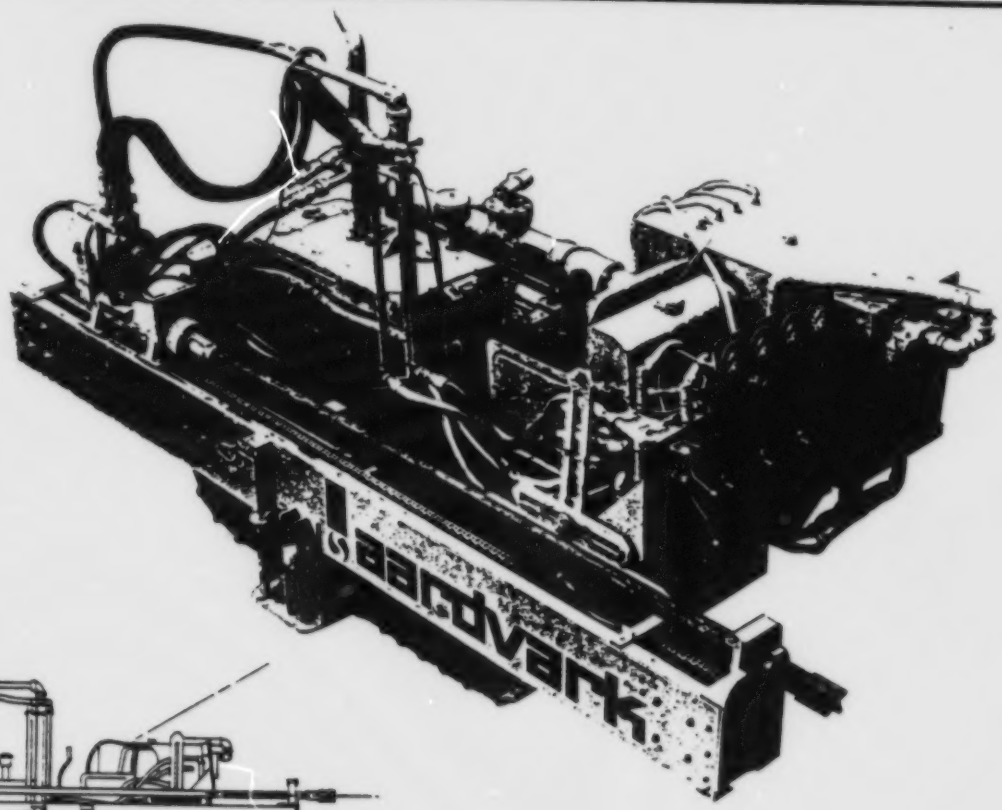
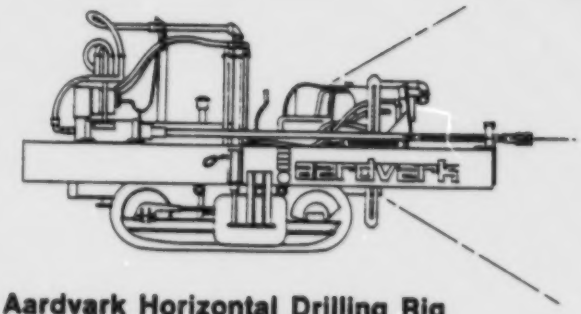
AARDVARK DRILL UTILIZED FOR UNDERGROUND DRAINHOLE DRILLING

FIGURE 5





Aardvark Horizontal Drilling Rig Performance Specifications

DRILL HEAD	Speed				250 r.p.m.	
	Torque				910 ft. lbs. @ 250 r.p.m.	
	Stroke				11 ft.	
	H.P.				43	
	Hyd. System Press.				2000 p.s.i.	
					f.p.m.	11 ft. stroke sec.
DRILLING RAM	Speed	In	Lo		0-33	0-20
			Hi		0-170	0-3.9
		Out	Lo		0-37	0-18
			Hi		0-190	0-3.5
	Hi Power	Out	Lo		0-16.5	0-40
			Hi		0-85	0-7.8
	Force	In			0-2950 lbs.	
		Out			0-2550 lbs.	
	Hi Power	Out			0-5500 lbs.	
	Hyd. System Press.				0-1000 p.s.i.	
WATER PUMP SYSTEM	Moyno Type 3L8	Capacity			92 g.p.m. @ 0 p.s.i. 78 g.p.m. @ 150 p.s.i. 835 r.p.m.	

Hydraulic motor drive at 2000 p.s.i. system pressure.

All above values calculated at 85% of theoretical values.

International Harvester TD-6 Crawler Tractor

ENGINE POWER	Horsepower, bare engine	94
	Horsepower, IEMC std.	79
	Rated Speed, r.p.m.	2300
TRACK FRAME	No. of Track Rollers, each side	5
	No. of Track Idlers, each side	2
	Track Shoe Width	16"
OTHER DATA	Crawler tractor is equipped with bi-metallic clutch, crankcase guard, heavy duty track and roller guards and fuel strainers.	
HYDRAULIC RESERVOIR CAPACITY		120 gals.
DRILL ANGLE	30° up and 30° down from horizontal. Carriage can be adapted to a forward pivot point for any vertical drilling angle through a range of 120°.	
DRILL ROD	3" o.d. 2.500" i.d. 10 l/f sections of alloy steel. Special drill rods, bits and subs available.	

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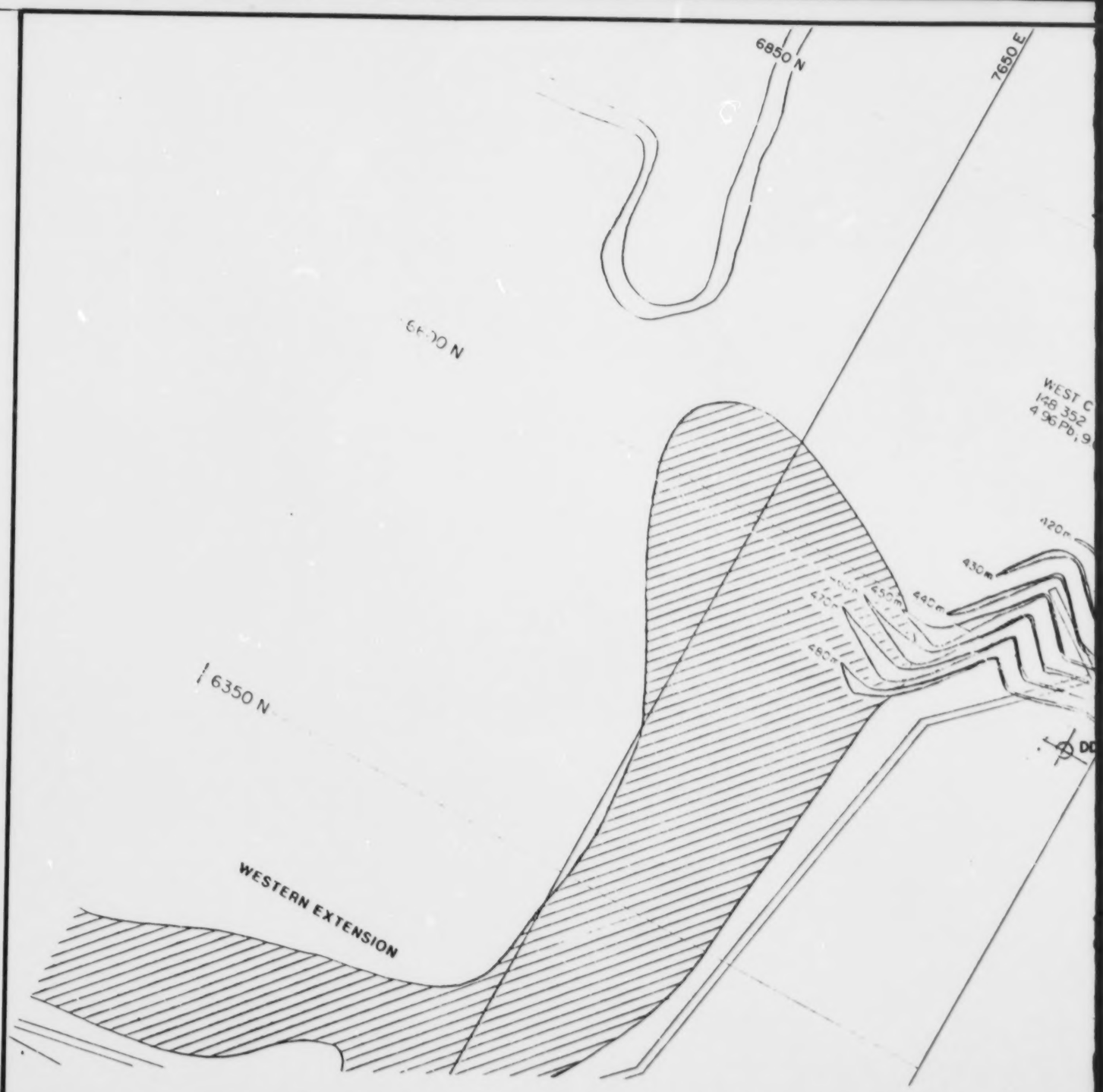
6850 N
7650 E
6750 N
WEST C
148 352
4.96 Pb, 9
1200
430m
440m
450m
6350 N
WESTERN EXTENSION

7900 E
7100 N
8150 E
8400 E
7350 N
8650 E
119 AREA
OW-1A
118 SUMP
PT-8
DW-1
118-853 DRILL STATION
BULKHEAD
118-1
118-3
PT-1
PT-10
118-844 DRILL STATION
842 X-CUT
846 X-CUT
DW-3
ETW-1
PT-5
PT-4
848 RAMP
851 DECLINE
PT-9
804 X-CUT
792 X-CUT
DDH-753
DDH-738
803 SUMP
PT-6
808 X-CUT
808/815 X-CUT
ETW-2
PT-3
PT-2
PT-11
PT-12
807 VENT RAISE
838 ESCAPE/VENT RAISE
OW-6
WEST C
8 352
6 Pb, 9.00 Zn
WEST B
68 333
5.46 Pb, 14.20 Zn
WEST A
6 522
3.80 Pb, 7.46 Zn
CENTRAL B ZONE
CENTRAL D ZONE
FORTAL
A
B
C
D
E
F
G
H-4

8650 E

8900 E

9150 E

9400 E

7850 N

7600 N

ROUTE 224

GAYS RIVER

CENTRAL E ZONE

CENTRAL C ZONE

CENTRAL D ZONE

CENTRAL G ZONE

CENTRAL G' ZONE

CENTRAL H ZONE

EASTERN B-1 ZONE

EASTERN A-2 ZONE

EASTERN B-2 ZONE

EASTERN C-2 ZONE

EASTERN C-1 ZONE

EASTERN A-1 ZONE

DW-1

OW-2

DW-2

OW-3

PT-7

H

PT-9

IOID SUMP

H-7

H-5

ETW-4

UNDE

Date..... NOV

Project..... 91

UNDERGROUND PRESSURE MONITORING AND AARDVARK DRILL LOCATIONS

FIGURE 6

8900 E
7600 N
9150 E
7850 N
9400 E
9650 E
9900 E
ROUTE 224
EASTERN B-1 ZONE
EASTERN C-2 ZONE
EASTERN A-2 ZONE
EASTERN B-2 ZONE
EASTERN A-1 ZONE
EASTERN C-1 ZONE
ETW-4
OW-3
H-7
H-5
7600 N

LEGEND

- AARDVARK DRAINHOLES (see tables 3 to 10 for details)
- PT-2 PRESSURE MONITORING POINTS (see tables 1 and 2 for details)
- SURFACE MONITORING HOLES
- (D) AARDVARK DRAINHOLE AREA (refer to text for discussion)

Date NOV 1991
Project 911 1721

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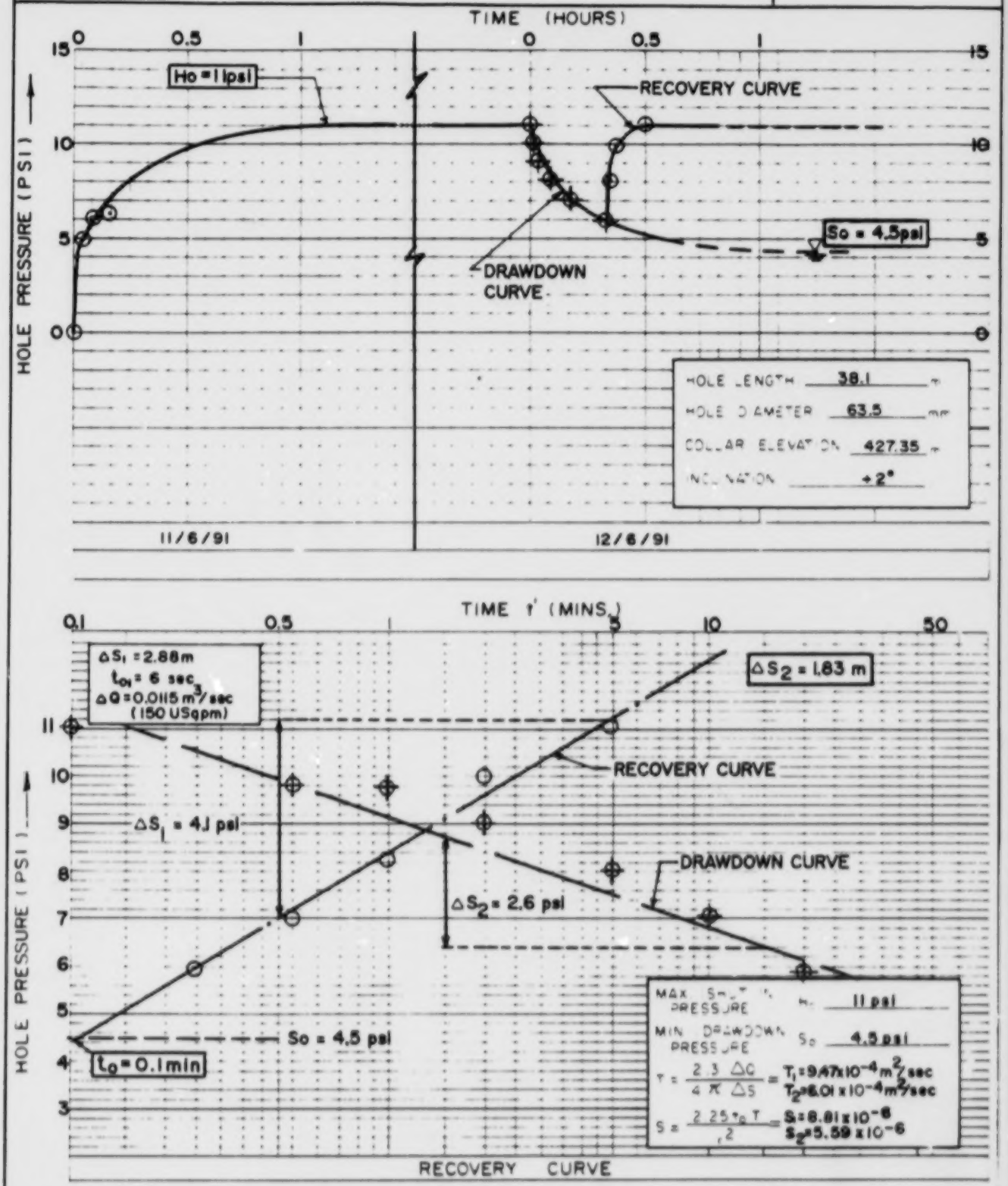
Chkd

DRAWDOWN AND RECOVERY HISTORY
(HOLE U649 - PT. 6 - 803 SUMP)

FIGURE 7


TIME (HOURS)
HOLE PRESSURE (PSI)
Ho = 11psi
RECOVERY CURVE
So = 4.5psi
DRAWDOWN CURVE
HOLE LENGTH 38.1 m
HOLE DIAMETER 63.5 mm
COLLAR ELEVATION 427.35 m
INCLINATION +2°
11/6/91
12/6/91
TIME t' (MINS.)
ΔS1 = 2.88m
t01 = 6 sec
ΔQ = 0.0115 m³/sec (150 USgpm)
ΔS2 = 1.83 m
RECOVERY CURVE
ΔS1 = 4.1 psi
DRAWDOWN CURVE
ΔS2 = 2.6 psi
MAX SHUT IN PRESSURE Ho 11 psi
MIN DRAWDOWN PRESSURE So 4.5 psi
T = 2.3 ΔQ / 4π ΔS = T1 = 9.47x10⁻⁴ m²/sec, T2 = 8.01x10⁻⁴ m²/sec
S = 2.25 t0 T / r² = S1 = 6.81x10⁻⁶, S2 = 5.59x10⁻⁶
So = 4.5 psi
t0 = 0.1 min
RECOVERY CURVE

Date NOV. 1991
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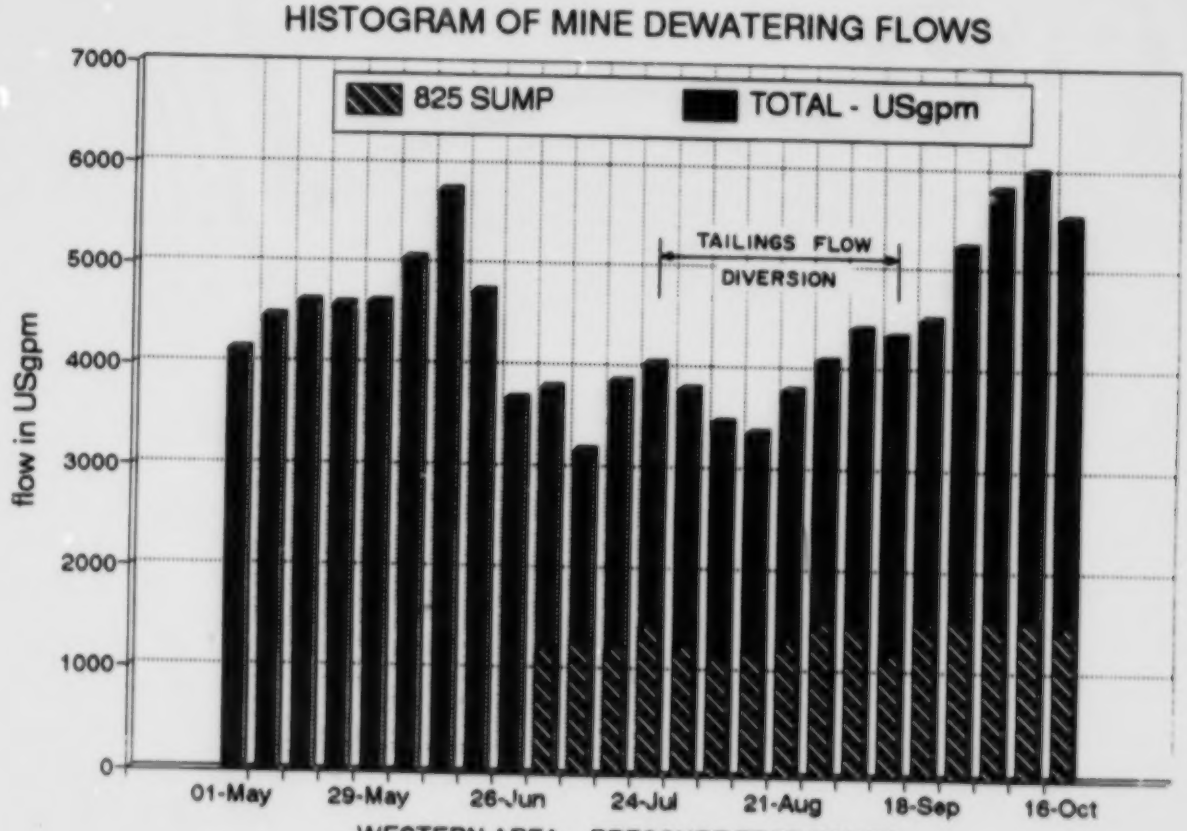
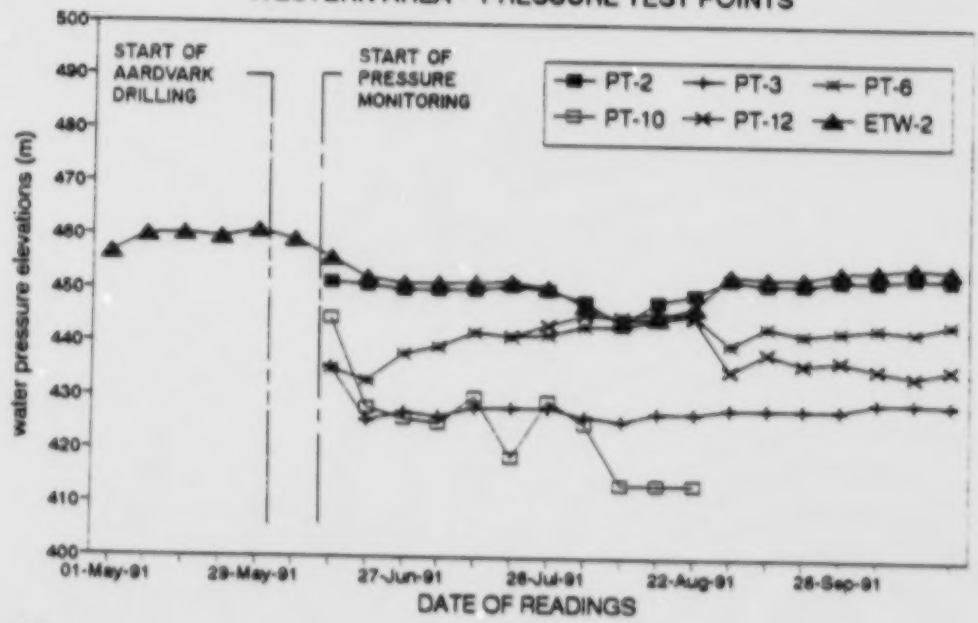
COMPARISON PLOTS OF FLOW AND PRESSURE BEHAVIOUR FOR UNDERGROUND DEWATERING FROM MAY to OCT. 1991

FIGURE 8





Date NOV. 1991

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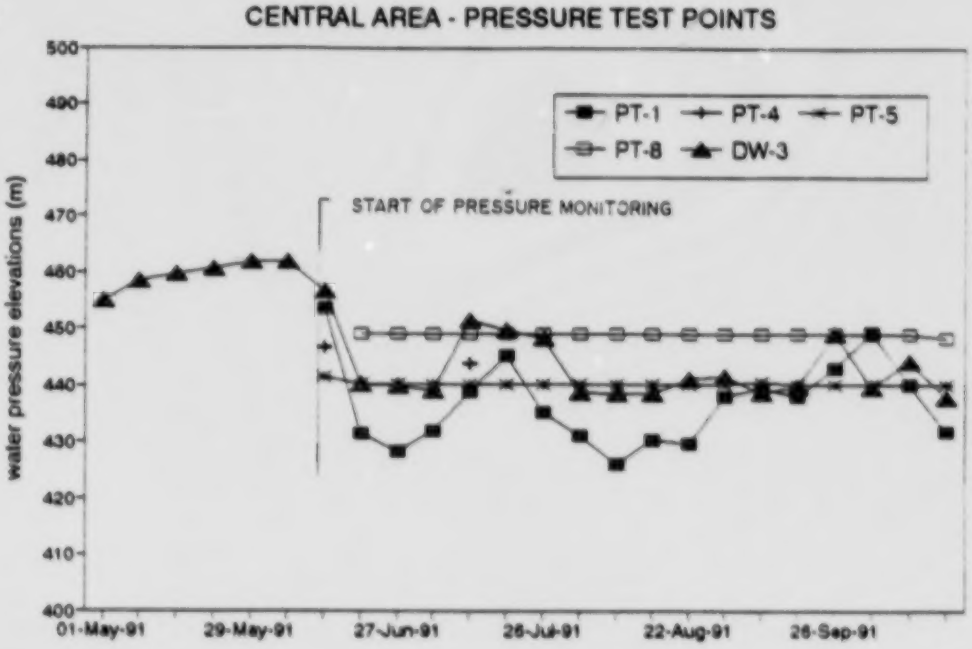
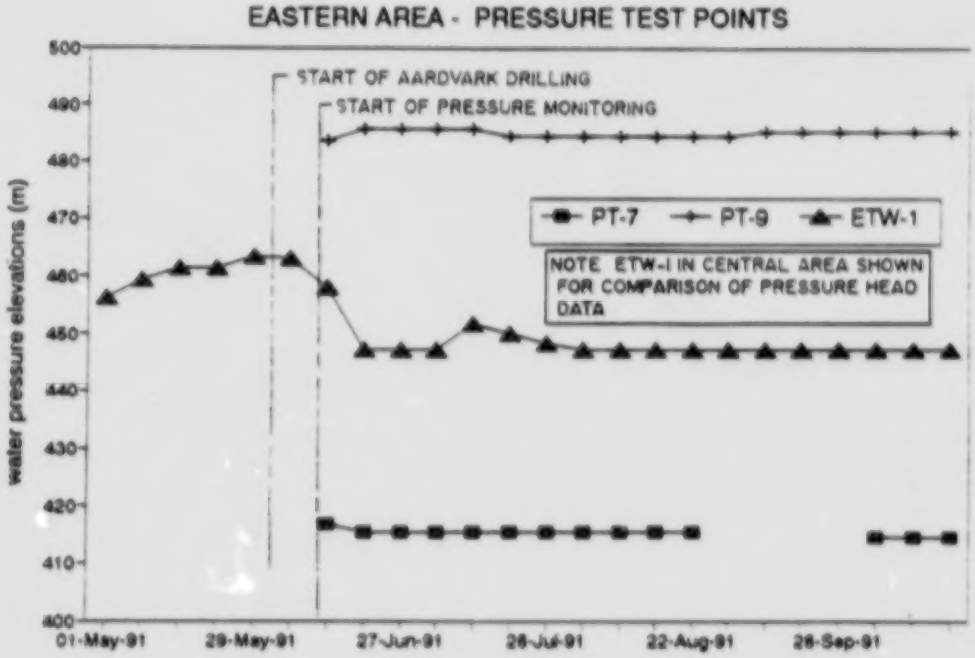
PRESSURE POINT MONITORING RESPONSE FOR PERIOD MAY TO OCT. 1991 (CENTRAL and EASTERN AREAS)

FIGURE 9





Date NOV. 1991

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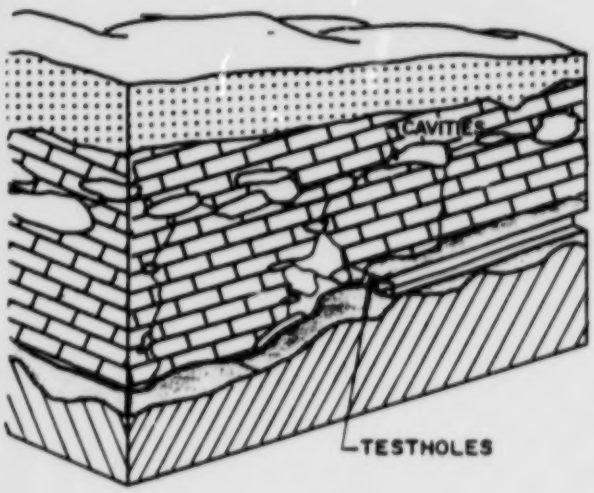
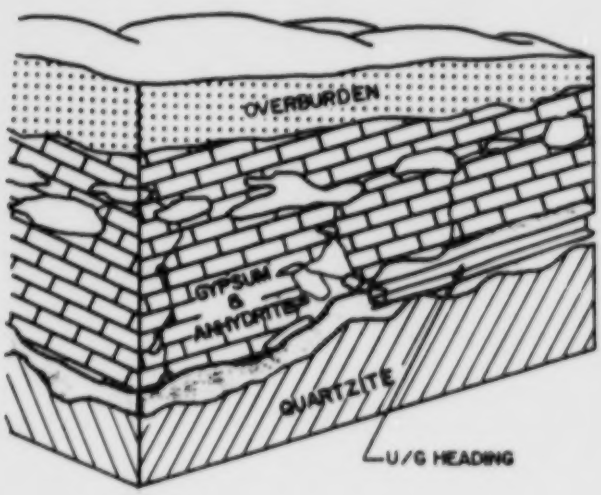
Chkd

POSTULATED SINKHOLE DEVELOPMENT ASSOCIATED WITH DRIFT EXCAVATION INTO KARSTIC ZONES

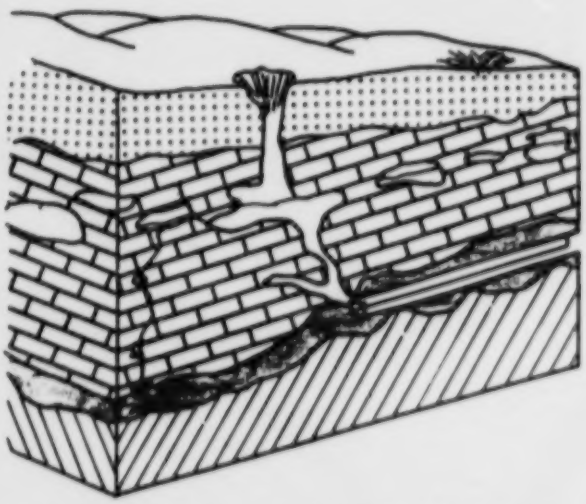
FIGURE 10



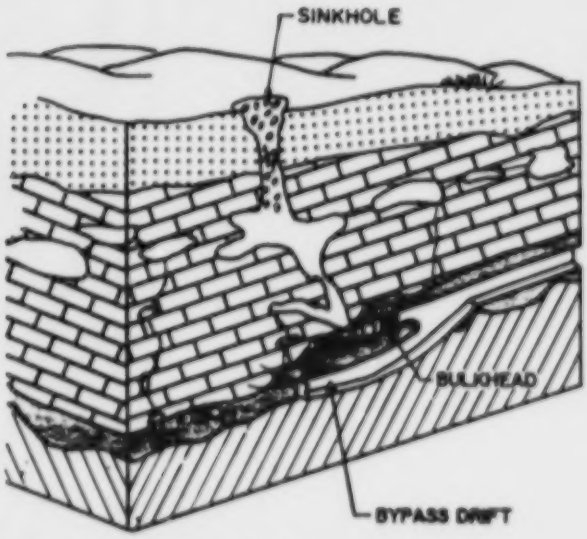
1. U/G HEADING ADVANCES IN ORE, TESTHOLING ALONG THE WAY. TESTHOLES MISS PIPING IN THE GYPSUM.



2. WATER BEARING KARST PIPE IN THE ROCK EXPOSED AFTER BLASTING.



3. HEADING FLOODS WITH WATER, SAND AND MUD AND A SINKHOLE FORMS ON SURFACE.



4. DRIFT IS PUMPED OUT AND REINFORCED CONCRETE BULKHEAD BUILT. A NEW HEADING REQUIRED TO BYPASS THE WATER.

SUGGESTED SEQUENCE OF PROBABLE MINING-INDUCED EVENTS BASED ON INTERNAL ESSO RESOURCES LTD. REPORT BY McKEE & HANNON, 1982

Date NOV 1991
Project 911-1721

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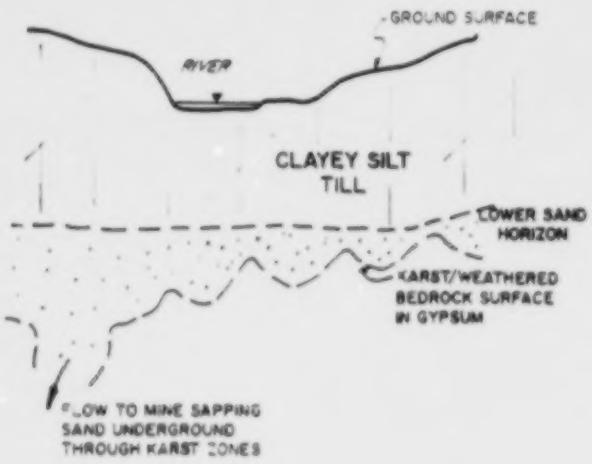
Chkd TAC

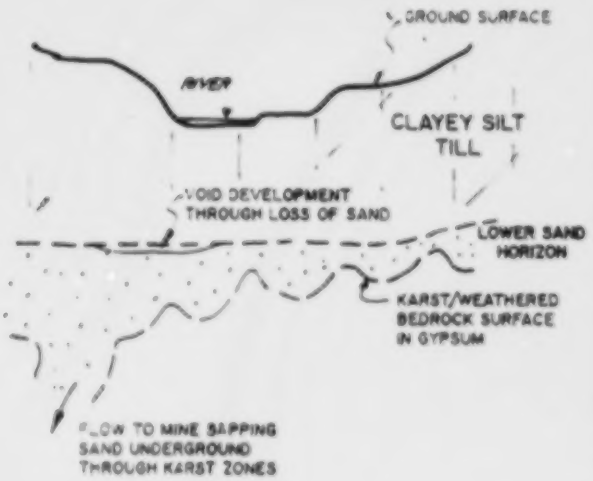
FORM PRODUCED JUNE 1988

POSSIBLE SEQUENCE OF SINKHOLE DEVELOPMENT THROUGH TILL

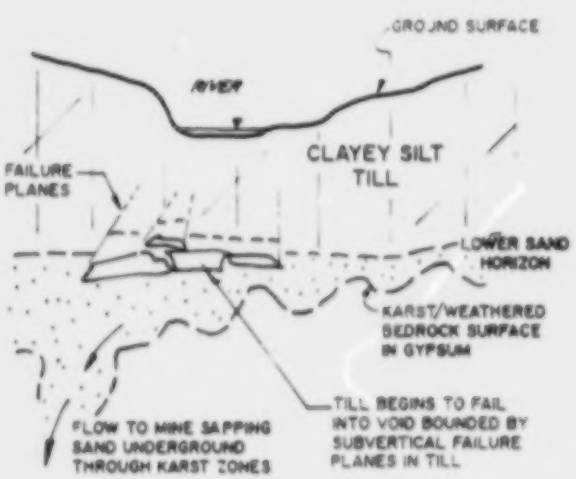
FIGURE 11



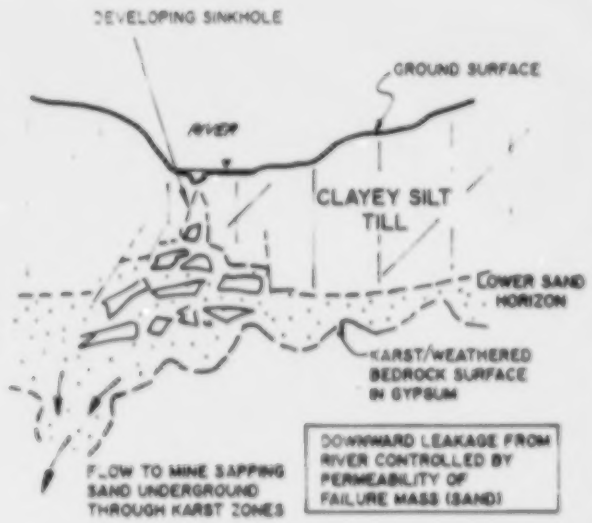
(1) INITIAL CONDITIONS


GROUND SURFACE
RIVER
CLAYEY SILT TILL
VOID DEVELOPMENT THROUGH LOSS OF SAND
LOWER SAND HORIZON
KARST/WEATHERED BEDROCK SURFACE IN GYPSUM
FLOW TO MINE SAPPING SAND UNDERGROUND THROUGH KARST ZONES

(2) INITIATION OF SINKHOLE DEVELOPMENT


GROUND SURFACE
RIVER
CLAYEY SILT TILL
FAILURE PLANES
LOWER SAND HORIZON
KARST/WEATHERED BEDROCK SURFACE IN GYPSUM
TILL BEGINS TO FAIL INTO VOID BOUNDED BY SUBVERTICAL FAILURE PLANES IN TILL
FLOW TO MINE SAPPING SAND UNDERGROUND THROUGH KARST ZONES

(3) SINKHOLE PROGRESSION (CRATER RETREAT)


DEVELOPING SINKHOLE
GROUND SURFACE
RIVER
CLAYEY SILT TILL
LOWER SAND HORIZON
KARST/WEATHERED BEDROCK SURFACE IN GYPSUM
FLOW TO MINE SAPPING SAND UNDERGROUND THROUGH KARST ZONES
DOWNWARD LEAKAGE FROM RIVER CONTROLLED BY PERMEABILITY OF FAILURE MASS (SAND)

(4) PROGRESSIVE DEVELOPMENT TO POINT OF SURFACE EXPRESSION IN RIVER

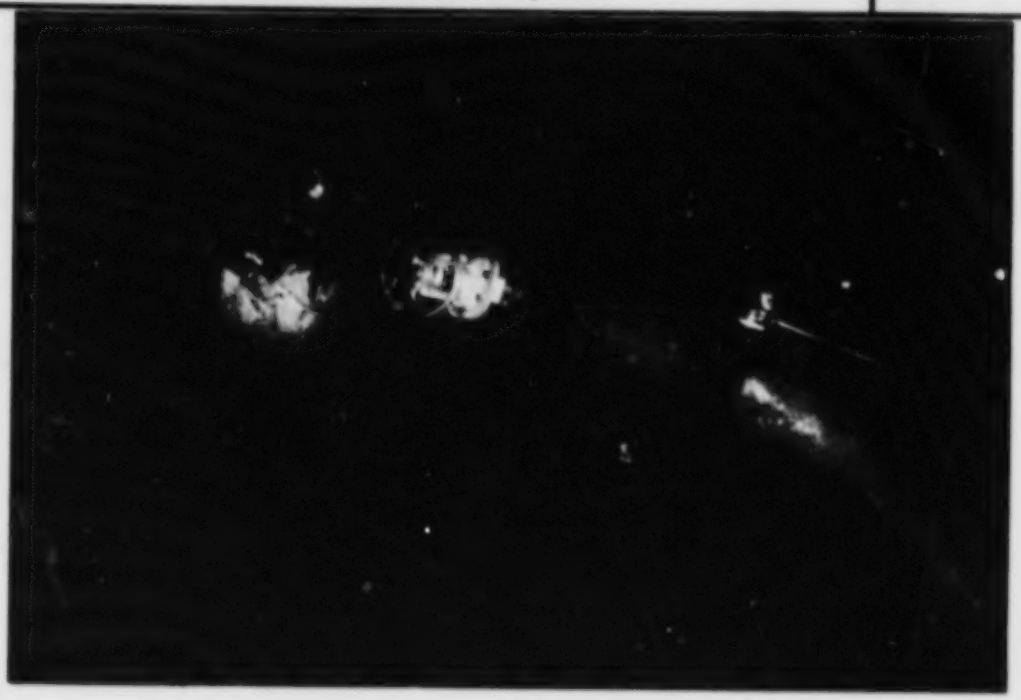
Date JULY 12, 1991
Project 911-1726

Golder Associates

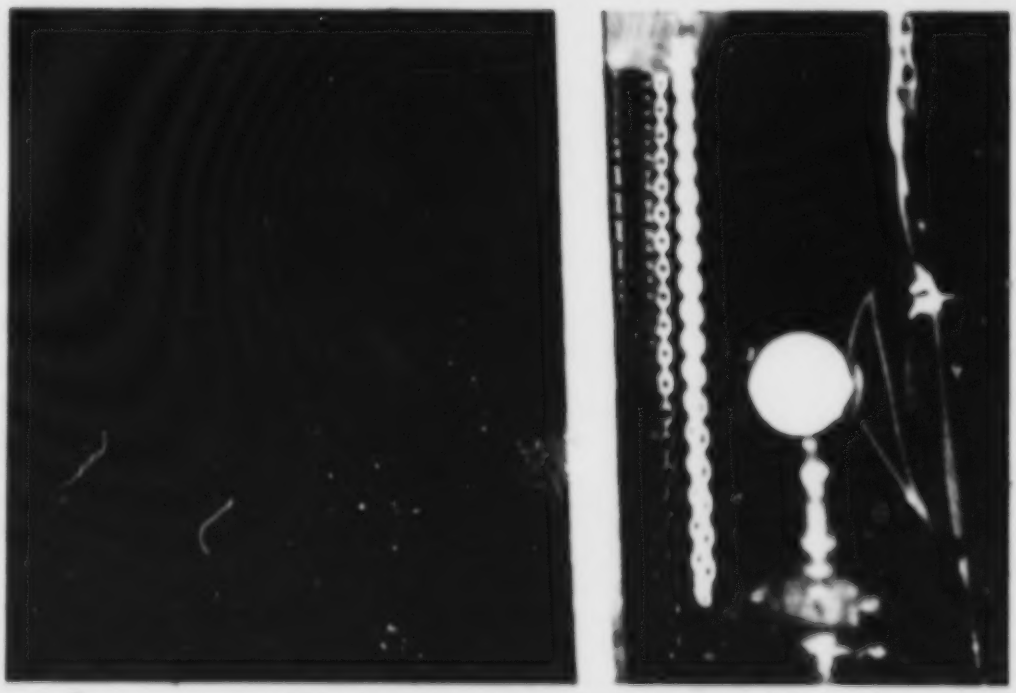
Drawn D.M
Chkd

CHARACTERISTICS OF FLOWS INTO 118 SUMP
(UNSCREENED HOLES)
PHOTOGRAPHED MAY 13, 1991

PLATE 1



OUTFLOW FROM UNSCREENED DRAINHOLE IN 118 SUMP AREA (c. 400 GPM)



TOTAL DEWATERING FLOW INTO 118 SUMP
(NOTE: SEDIMENT CONCENTRATION IN WATER)

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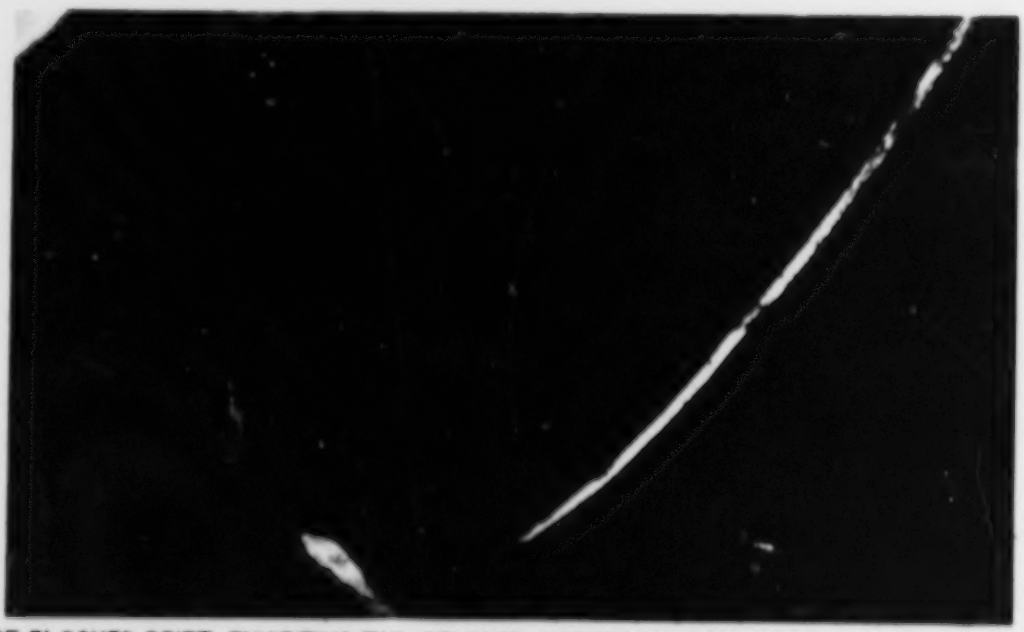
PHOTOGRAPHS OF TRENCH MATERIAL AS EXPOSED INSITU AND AFTER INFLOW CONDITION

PLATE 2

CHARACTERISTIC GEOMETRY OF ORE ZONE / TRENCH MATERIAL CONTACT BOUNDARY



VIEW OF BLOCKED DRIFT, EXHIBITING TAIL OF SAND / MUDFLOW FROM STOPE IN THE 848 RAMP AREA

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APPENDIX A

MEMORANDA RELATING TO DEWATERING ASPECTS

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Mississauga, Ontario
L5N 5S3

Tel: (416) 567-4444 Fax: (416) 567-6561

FACSIMILE TRANSMISSION

To:	Westminer (Canada) Ltd.
Attention:	Tom Lamb
Fax No:	(902) 883-9245
From:	T.G. Carter
Date:	June 3, 1991
Project No:	911-1721
RE:	SUGGESTED WATER FLOW MEASUREMENT TECHNIQUES ETC.

Total number of pages (including cover page): 18

Message:

Attached please find a

(i) paper on grouting of a problem inflow zone (for your information re: some good reasons for not drilling drainholes collared in the evaporate - as if you lose control by solutioning around the valve housing, you may never be able to regain control),

(ii) some thoughts on the types of procedures that we should be thinking about for testing individual and multiple holes (as follows):

Suggested Approach

i) Make sure pumping capacity is adequate to cope with flows:

ii) Monitor pressure and flow conditions at several of the accessible underground drillholes to attempt to determine hydraulic response across the mine section. Attempt to refine current drawdown information.

a) Set up a wide section of holes with permanent pressure gauges and a second ball valve as per sketch (attached).

b) Start to record pressure at underground holes on weekly basis. Continue to monitor groundwater levels from available surface holes.

Conduct Field Tests

i) During drilling of future holes through fracture zone into trench, record drillwater flush returns and strange water characteristics.

ii) For any proposed cored holes carry out RQD, TCR and SCR percentage logging, fractures/foot should also be recorded. Intention of this is describe width of controlling fractures and/or vuggy zones and assess rock conditions in inflow zone - is it margin? is it vuggy carbonate? Where is the main inflow? For percussion holes, record penetration rates and chip recovery to attempt to keep accurate log of geology.

iii) Leave characterization holes open and valved complete with T piece and pressure gauge (0-200 psi or suitable) (see sketch).

iv) When no other activity (eg. drilling) underway in the immediate vicinity of individual holes carry out water pressure and recovery tests.

Test procedure is as follows:

Once hole drilled and free-running, make measurements of pressure regime based on data from individual holes. It is important that records of flow and pressure be determined from the various holes. (This is best done by quickly turning off valve - read pressure - turn water back on, leave for a few minutes then fill a 25 litre bucket - measure the time in seconds to fill the bucket).

For the new, free flowing holes, readings should be taken on day 1 at 15 minutes after the hole has been drilled and successfully valved, then at 30 minutes; 1 hour; 2 hours; 4 hours; 8 hours; 16 hours and on day 2 at 32 hours after first drilling and valving the hole.

After completing this (drainage phase) test, have enough people ready to monitor pressures on each hole in the area, then try repressurization phase, test by closing the valve on one of the new fast-free-flowing holes.

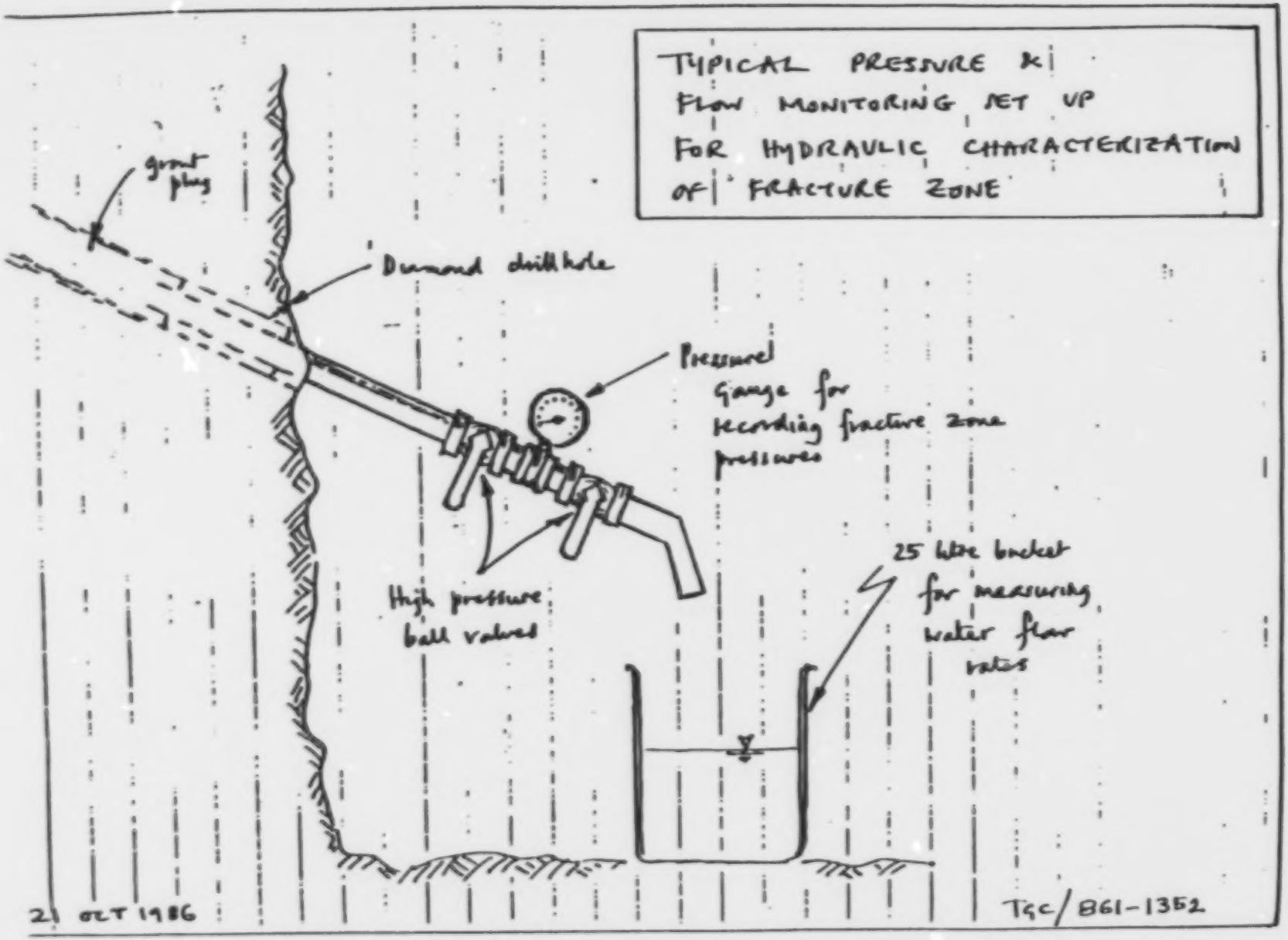
Pressures on the pressure gauges on <u>all</u> monitored holes in the area should be recorded at the following times after closing the valve: 15 seconds (i.e. as close to immediately as possible), then at 1 minute, 2 minutes, 5 minutes, 10 minutes, 20 minutes, 45 minutes, 1.5 hour, 3 hours, 6 hours, 12 hours, 24 hours, 48 hours.

Ideally throughout all this period, no other new drainage holes should be put in or perturbation of the flow in the trench/fracture zone(s) should be permitted (i.e. no drilling or other activity that could affect the test results). Operational constraints may necessitate some compromise to this, however.

Some graphs and data tabulations from these types of test are attached as are some details for V-notch weir approaches for better quantifying underground flows from different areas.

These types of test will allow some better appreciation of head response/recovery behaviour within the trench/rock mass system. The approach is crude and simple, but probably the best we can do meaningfully.

TGC/se


TYPICAL PRESSURE & FLOW MONITORING SET UP FOR HYDRAULIC CHARACTERIZATION OF FRACTURE ZONE
grout plug
Diamond drillhole
Pressure gauge for recording fracture zone pressures
High pressure ball valves
25 litre bucket for measuring water flow rates
21 OCT 1986
TGC/861-1352

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Project: _____ DRILL HOLE DATA

LEVEL No. 150 metre — COLLAR ELEVATION: 5150 — ANGLE : -50° (UP) (DOWN)

HOLE No. MUG 150-14 — STATIC (SHUT IN) PRESSURE: 221 psi — (P.S.I.) 155 (m) (H_o)

HOLE LENGTH 119.8 m — MIN. DRAWDOWN PRESSURE: 20 psi — (P.S.I.) 14 (m) (S_o)

DATE	CLOCK TIME	ELAPSED TIME Min			PRESSURE		DRAWDOWN	RECOVERY (S_o-S)	RESIDUAL DRAWDOWN (S')
	H:Min.	t	t'	t/t'	(P.S.I.)	m (H)	(H_o - H)	(H - S_o)	(H_o - H)
18/01/89	12:41	0			221	155			
115 L/min									
	12:42	0			221	155			
47 L/min	12:45	3			80	56	99 m		
31 L/min	13:00	18							
16 L/min	14:00	78			20	14	141 m		
Recovery	14:07	85	0		20	14	141 m	0	141
	14:07:15	85.25	.25		46	32		18	123
	14:07:30	85.5	.5		57	40		26	115
	14:08	86.	1		70	49		35	106
	14:09	87	2		88	62		48	93
	14:10	88	3		100	70		56	85
	14:11	89	4		110	77		63	78
	14:12	90	5		119	83		69	72
	14:18	96	11		150	105		91	50
	14:23	101	16		167	117		103	38
	14:37	115	30		195	137		123	18
115 L/min	15:07	145	60		211	148		134	7

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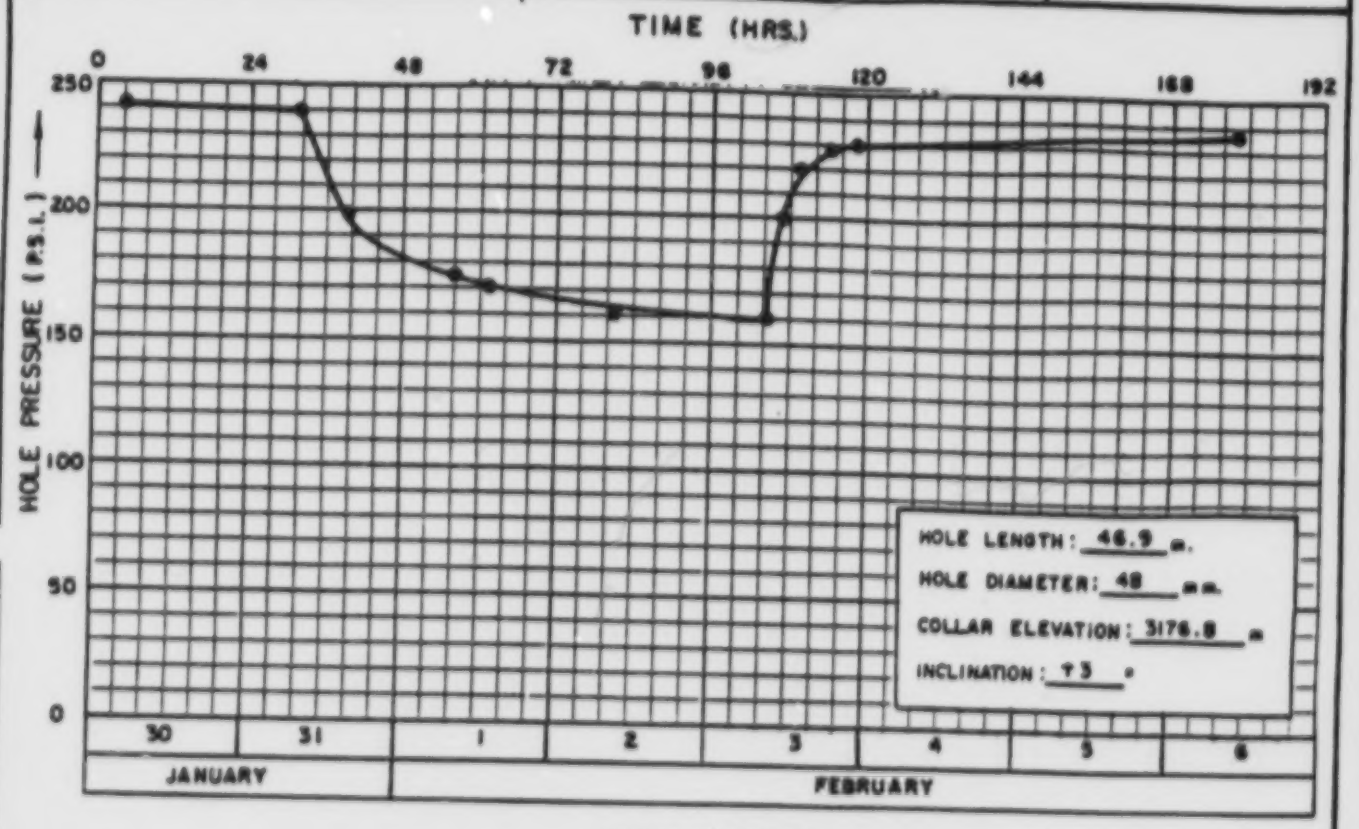
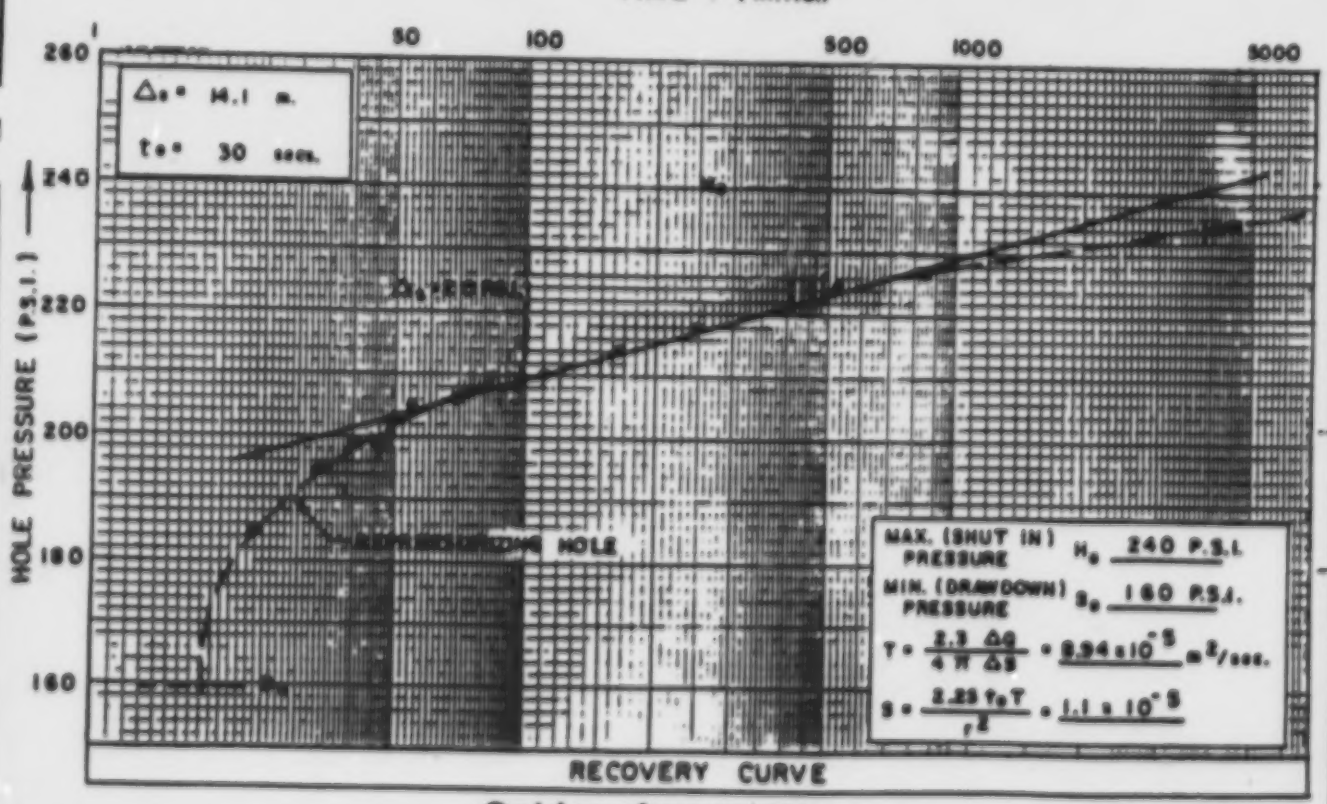
DRAWDOWN AND RECOVERY HISTORY
DRILLHOLE C-1617 (3-05 DRIFT)

FIGURE 15A





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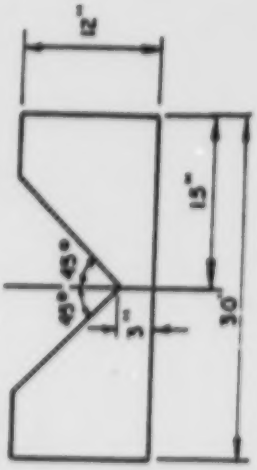
DETAILS FOR 90° V-NOTCH WEIRS

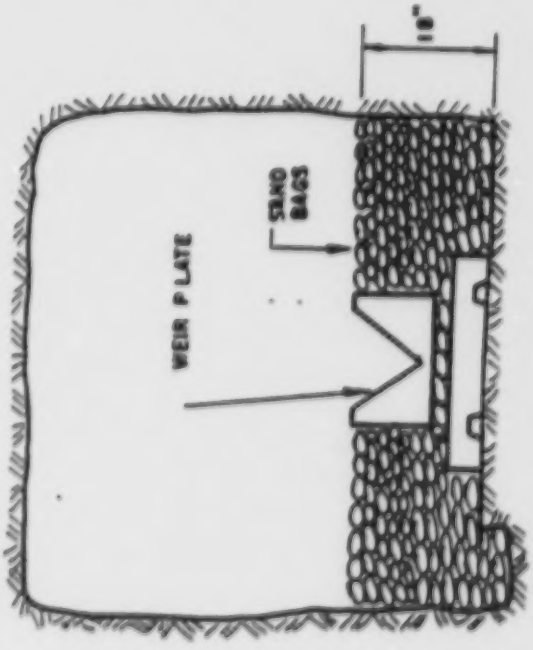
FIGURE 8

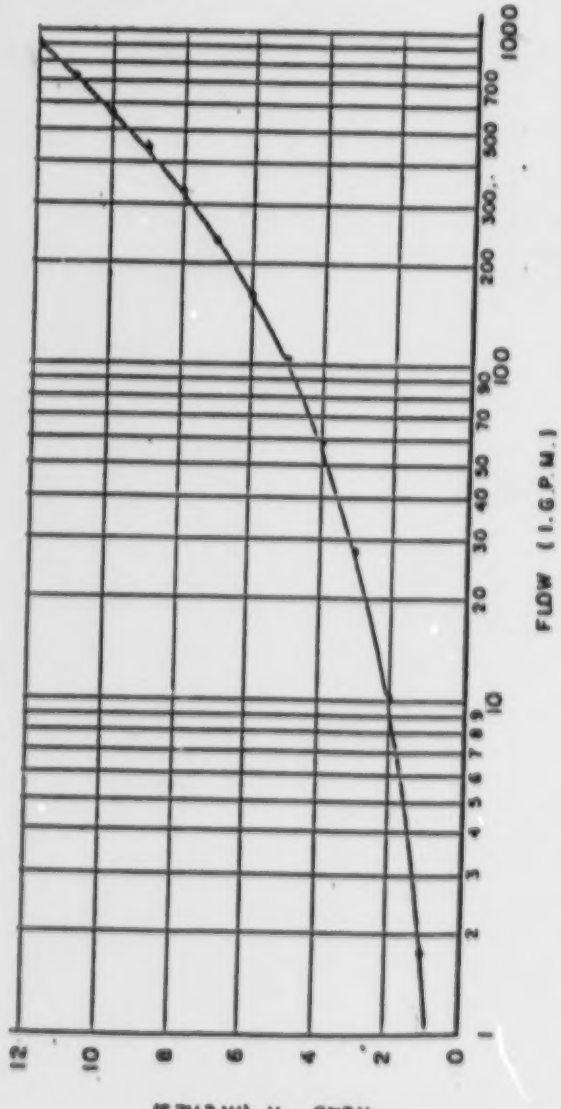
HEAD INCHES	FLOW I.G.P.M.
1	1.8
1.25	3.2
1.5	5.0
1.75	7.4
2	10.3
2.25	13.9
2.5	18.1
2.75	22.9
3	28.4
3.25	34.8
3.5	41.9
3.75	49.8
4	58.5
4.25	68.1
4.5	78.5
4.75	90
5	102
5.25	115
5.5	130
5.75	145
6	161
6.5	197
7	237
7.5	282
8	330
8.5	386
9	444
9.5	508
10	578
10.5	653
11	733
11.5	820
12	912



α = CUT ANGLE = 90°
g = 32.2 ft./sec.²
C = FRICTION FACTOR = 0.57
H = HEAD (in.)
Q = FLOW (I.G.P.M.)

$$Q = C \cdot \theta \cdot \frac{H^2}{15} \cdot \sqrt{2gH} \tan\left(\frac{\alpha}{2}\right)$$





Date FEB. 22, 1984
Project No. 851-1215

Golder Associates

Drawn D.M.
Chkd.

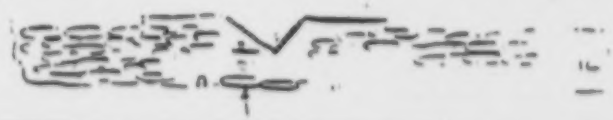
PROJECT: _____ No. _____

LEVEL: _____

WEIR No. _____

FLOW : DATA

SKETCH OF WEIR PLATE GEOMETRY



DATE	CLOCK TIME	ELAPSED TIME	DISCHARGE DATA			BUCKET & STOPWATCH CHECK		
			OVERFLOW HEIGHT		FLOW	QUANTITY	TIME	FLOW
	H : MIN.	(MINUTES)	(ins)	(cm)	Igpm.	(ℓ)	MIN:SEC.	ℓ/MIN.
26/Jan	8:30		1 1/4		5.2			
27 Jan	8:30		1 1/2		5.2			
31 Jan	8:22		1 1/4		5.2			
	10:15		2 1/8		8.8			
	16:30		2 1/2		13.9			
	11:40		2 1/2		13.9			
	13:00		2 3/8		16.0			
	17:45		2 1/2		13.9			
1 Feb	8:07		2 1/8		8.8			
	9:15		2 1/8		8.8			
2 Feb	8:12		2 1/8		8.8			
3 Feb	8:14		2		8.3			
	12.42		1 1/8		5.2			

PROJECT _____ No. _____ DRILL HOLE DATA

LEVEL No. _____ COLLAR ELEVATION: _____ ANGLE : (UP) (DOWN)

HOLE No. _____ STATIC (SHUT IN) PRESSURE: _____ (P.S.I.). (m) (H_o)

HOLE LENGTH _____ MIN. DRAWDOWN PRESSURE: _____ (P.S.I.) (m) (S_o)

DATE	CLOCK TIME	ELAPSED TIME			PRESSURE		DRAWDOWN	RECOVERY (S_o - S)	RESIDUAL DRAWDOWN (S')
	Hr:Min.	t	t'	t/t'	(P.S.I.)	m (H)	(H_o - H)	(H - S_o)	(H_o - H)

DRAWDOWN AND RECOVERY HISTORY

FIGURE

TIME (HRS.)

HOLE PRESSURE ( ) ⟶

HOLE LENGTH : _____ m.

HOLE DIAMETER : _____ m m.

COLLAR ELEVATION : _____ m

INCLINATION : _____ °

TIME t' (MINS.)

1 3 10 30 100 300

HOLE PRESSURE ( ) ⟶

Δs = m.

t_0 = secs.

MAX. (SHUT IN) PRESSURE H_0 _____

MIN. (DRAWDOWN) PRESSURE S_0 _____

$T = \frac{2.3\ \Delta Q}{4\pi\ \Delta S}$ = _____ m^2/sec.

$S = \frac{2.25\ t_0 T}{r^2}$ = _____

RECOVERY CURVE

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FACSIMILE TRANSMISSION

To:	Westminer Canada Ltd
Attention:	Tom Lamb
Fax No:	902-883-9245
From:	Trevor Carter
Date:	June 5, 1991
Project No:	911-1721
RE:	Flov Rates etc.,

Total number of pages (including cover page): 5 nev + 18 old

Message:

Attached are two sets of graphs for 40ft and 60ft long holes for various applied heads for two sets of roughnesses - e=1mm and e=3mm. You can see the effect of the roughness increase from 1 to 3mm by comparing the curves for the different head configurations. For the Hole 808 Ring2-01 with a 36m head (51.5psi), 4.5" hole diameter, the initial flov should have been in the 1200 to 1500 gpm. range. With a 1.5" hole size you can see the throughput flov drops incredibly - to of the order of only 200 gpm maximum.

Several questions arise - did the flov drop on the hole because
a) the screen collapsed
b) the screen blocked off with fines or simply
c) the flov is nov only coming through the screen & pipe

With a 10 slot screen and a limited hole size the entrance area is much reduced, thus flovs will drop drammatically. Graphs are available which allov these things to be assessed.

You should establish whether the screen is open still right to the end of the hole - try an air-vater jet lance pushed right to the back of the hole within the screen. If the screen is still functioning but has trapped all the sand, then the maximum flov can be rapidly estimated from the folloving:

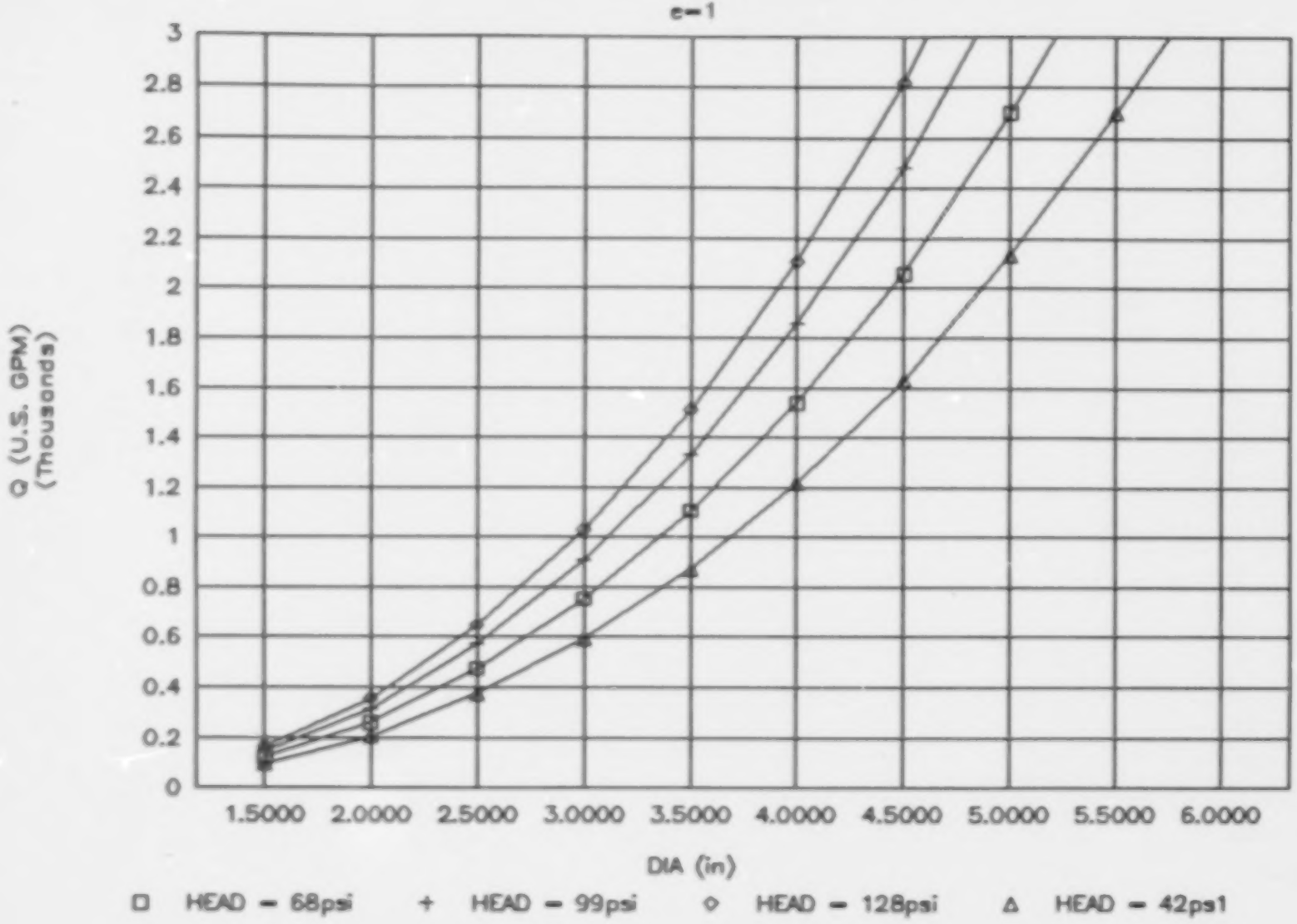
Q (Igpm) = $(L_\omega \times P \times L)/(2150)$ approx.
where L = Length of Test/Screen Section (ft).
P = Applied head on test section (psi)
L_ω = Effective Permeability of section in Lugeon Units
NB: 1 Lugeon = 10^{-5} cm/sec.

For sand, say 10^{-3} is a good number, = 100 Lugeons, therefore, flov would drop to only about 35 Igpm! Does this make sense in the actual situation - You should be able to check out the screen and see if its broken.

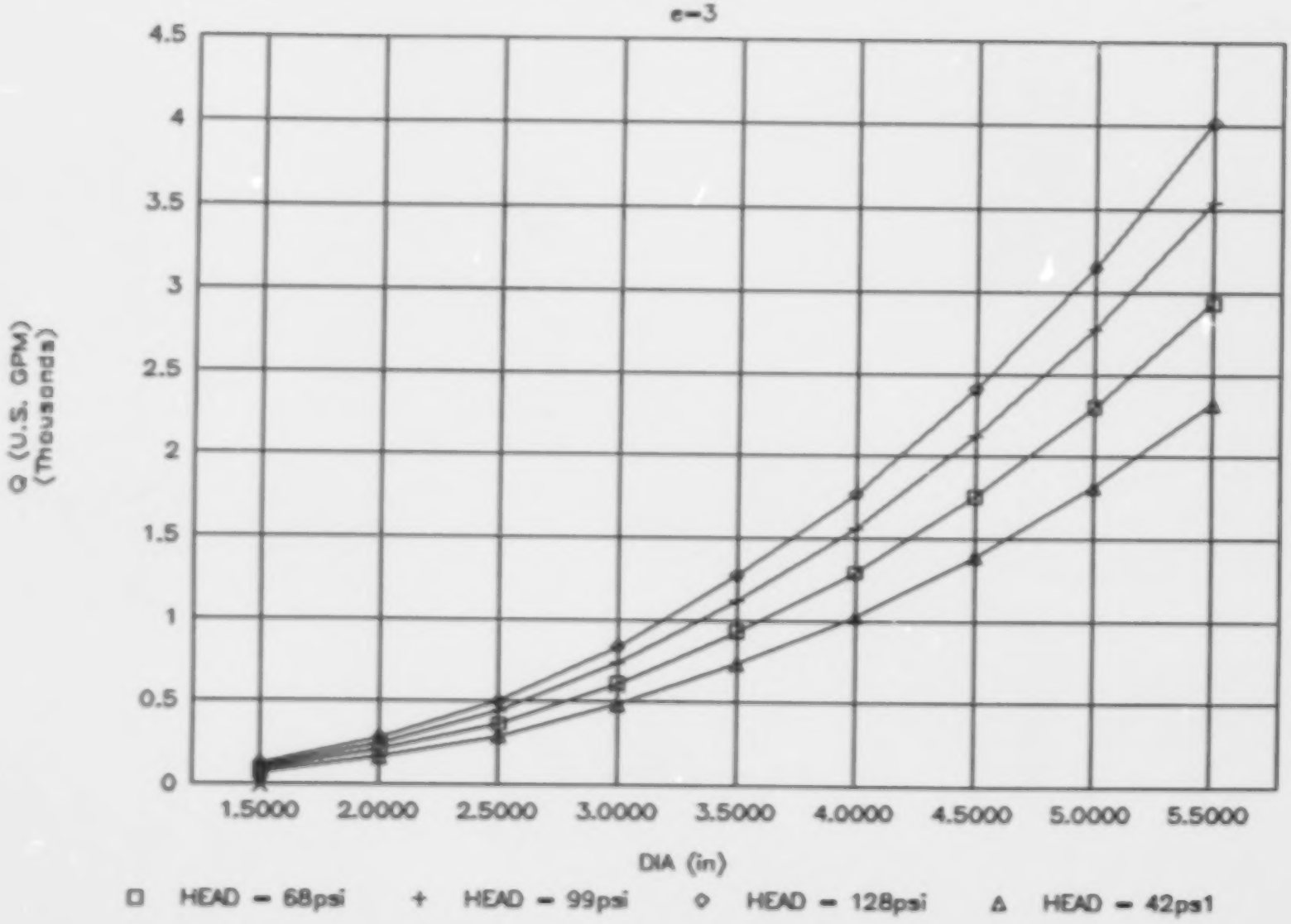
If you go with a 50 slot screen, this vill likely pass a lot of the sand + fine fraction initially until a coarse graded filter naturally develops around the vell head, at vhich point the entrance velocities at the outer periphery of the "gravel pack" that has been created in the *Trench Zone* vill be small and no more sand will pass. Nevertheless it is vorth trying. 20 slot may be a better compromise but this will again reduce the throughflov/output from the hole. Thoughts would be - drill 4.5" hole and case to the end of the Goldenville quartzites then put telescoped smaller screen in from this point to and through into the production zone. This vould give maximum advantage of big hole flov, vould minimize screen length and hopefully maximize flovs.

Re: our conversation on screen selection - Paul Puodzuinas in our office here is our vell man, he could give you some extra help or contact Johnson directly - Don Baron 517-337-1124 in the States.

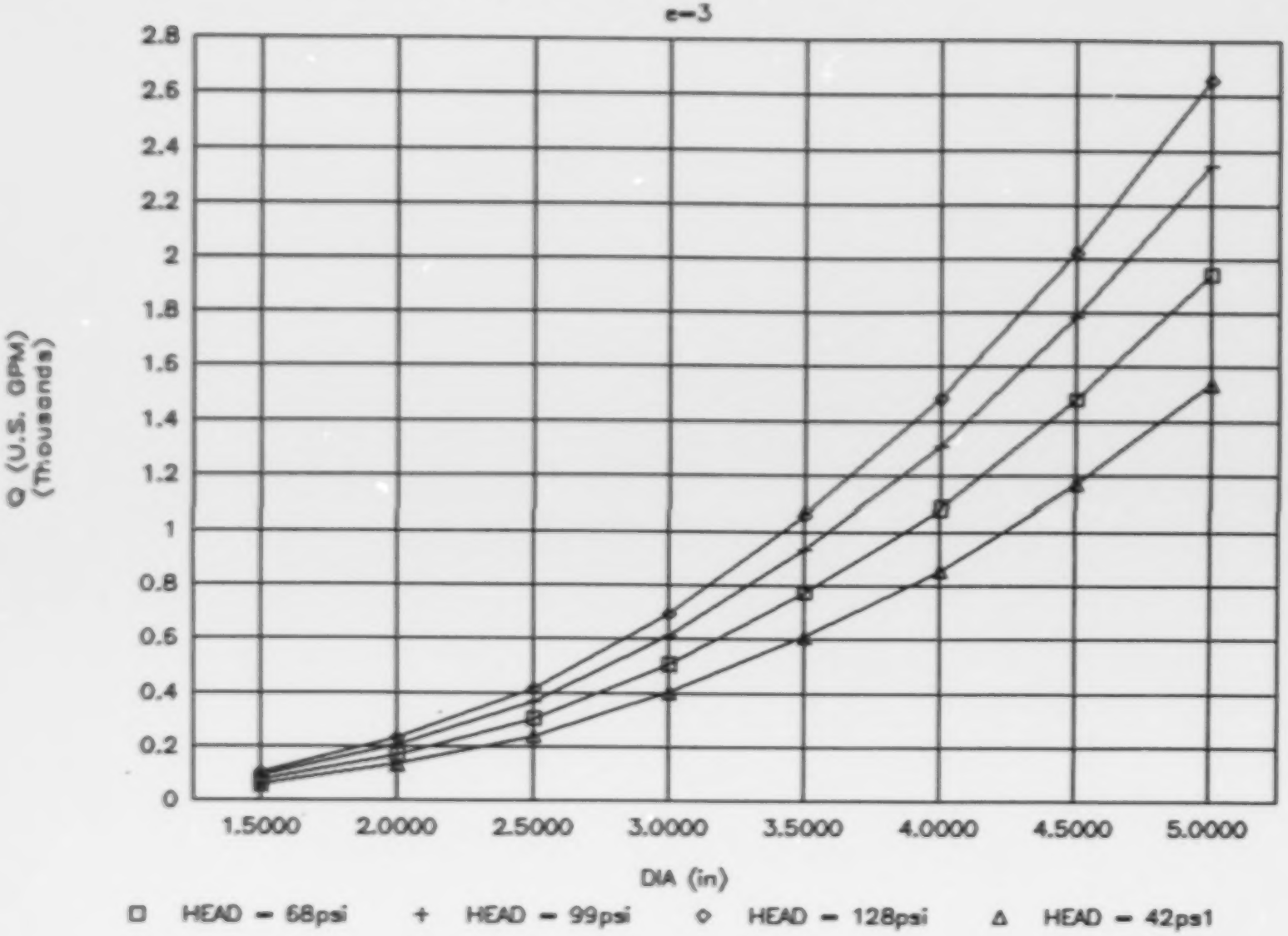
HOLE LENGTH = 40'



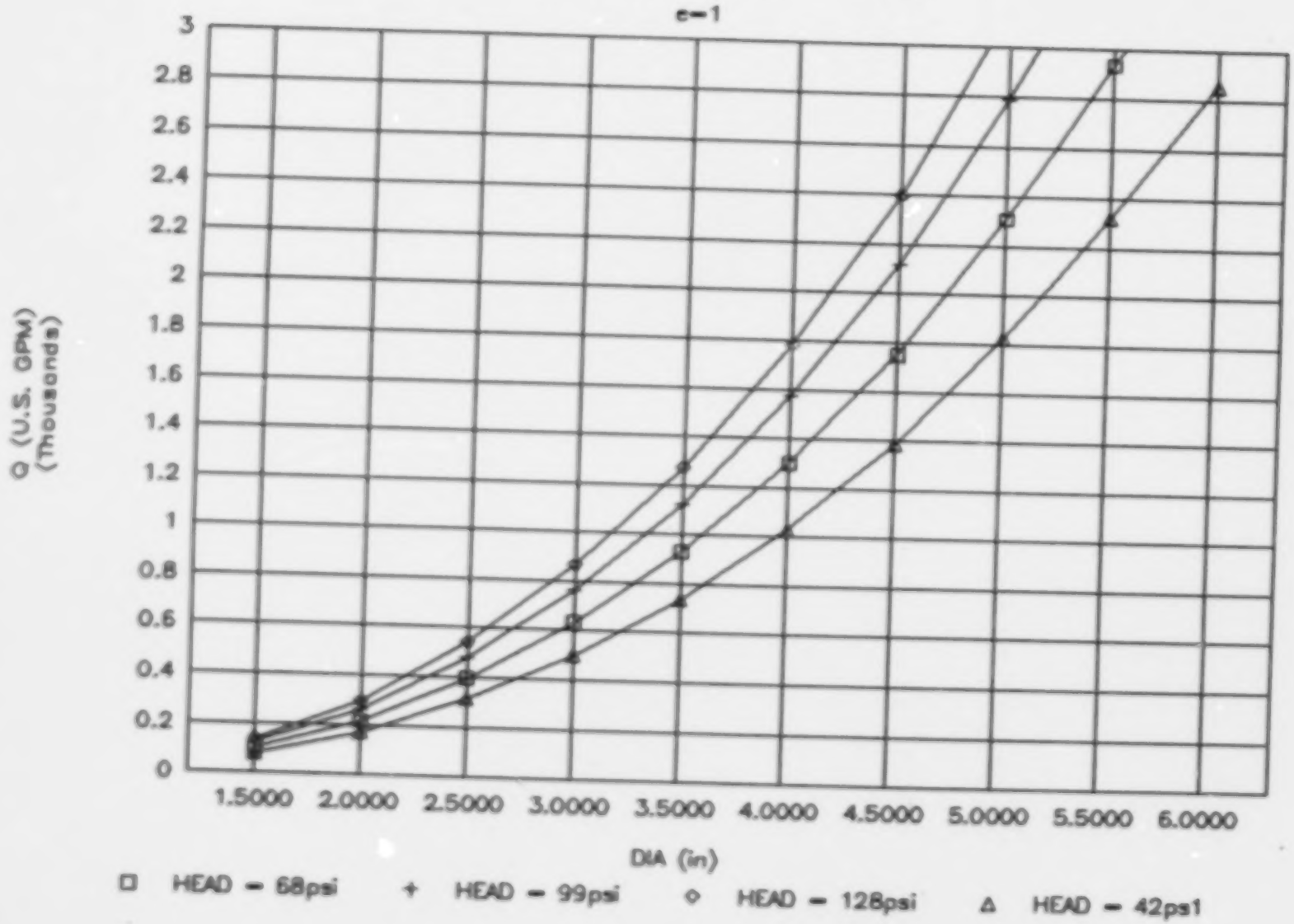
HOLE LENGTH = 40'



HOLE LENGTH = 60'



HOLE LENGTH = 60'



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FACSIMILE TRANSMISSION

To:	Tom Lamb, Joe Campbell
Organization:	Westminer Canada Ltd
Fax No:	902-883-9245
From:	Trevor Carter / Rob Blair
Date:	July 16, 1991
Project No:	911-1721
RE:	Gypsum Drift - Aardvark Drilling - Considerations for Problematic Solutioning

Total number of pages (including cover page): 9 (5 text + 4 figures)

Aardvark Drilling

Have just received the attached article - in particular, see last paragraph on page 137 and top paragraphs on page 138, attached - it suggests we should be really careful with regard to flow solutioning:

a) around the casings for the aardvark holes collared in the evaporites
b) globally within the evaporite units once significant flows are allowed to pipe down the aardvark holes

With the fact that there are known minor leaks under 100 psi. pressure around the casings, I suggest that serious thought be again given to constructing a concrete bulkhead in the drift within which flanges for the pipe casings can be set and through which tie-back anchors can be installed and the whole structure sealed by secondary grouting - see attached sketch - Figure 1.

On the global front - depending on what "effective thickness" one considers is responsible for the flow to the Pressure Monitoring Holes - the pressure-flow response data suggests hydraulic conductivities in the 10^{-3} to 10^{-4} cm/sec. range for most of the higher flow holes - these are in the same order of magnitude as those reported in the paper by O'Neil and Gourley, 1991 - the gypsum solution rates therefore could be comparable!

Conceptual Model

Further to yesterdays fax, attached on Figure 2 is a sketch of the trench zone showing a possible scenario to explain the *measured pressures* shown on the scrolled-out section presented on Figure 3. A comparison to published understanding of classic Karst internal structure is shown on Figure 4.

Best Regards,

Trevor

Golder Associates

Bulletin of the
Association of Engineering Geologists
Vol. XXVIII, No. 2, 1991
pp. 127-143

Geologic Perspectives and Cause of the Quail Creek Dike Failure

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Alan L. O'Neill, Inc., 1058 Buchan Drive, Lafayette, CA 94549

CHAD GOURLEY
Utah State Engineer's Office, 1636 W. North Temple Street, Salt Lake City, UT 84116

ABSTRACT

January 1, 1989 was marked by the catastrophic failure of Quail Creek Dike, located near St. George, Utah. The dike was a 24-m (78-ft) high, 604-m (1,980-ft) long embankment constructed in 1984. Several episodes of increased seepage, discolored seepage, and small sinkholes occurred in 1986, 1987, and 1988. The downstream area was evacuated in time to avoid fatalities, however the damage was estimated in the millions of dollars including disruption of major highways and bridges.

Quail Creek Reservoir is an offstream storage project for water supply and hydroelectric power. The project is owned and operated by the Washington County Water Conservancy District.

Quail Creek Dike was constructed across the crest of a doubly plunging anticline in thin-bedded sedimentary rock containing gypsum-rich beds. After the failure, gypsum content was found to be as much as 50 percent in some beds. The combination of structural effects caused by arching of the anticline and the presence of significant amounts of soluble salts presented a complex and challenging foundation for the dike.

Participation of an engineering geologist was limited during exploration, design, construction, and operations phases of the project. State-employed geologists reviewed some phases of the work, but their ultimate responsibility in the final design approval process and for foundation approval during construction was not clear.

Post-failure investigation revealed a foundation that was strongly jointed and with discrete seepage paths particularly near the surface. Several factors contributed to the failure including: a) lack of understand-

greatest flexure. Extensional fractures of this type posed several problems. Solution cavities and grout infillings, as observed in the breach, suggest that the extensional fractures were substantially open, as might have been expected, prior to the breach. Further evidence of open fractures was the immediate appearance of seepage upon first reservoir filling. Rock units examined in the breach area exhibited different joint patterns with both orientation and density varying between units (Figure 11). Joint spacing ranged from less than 1 joint per 3 m (10 ft) in less competent units to more than 4 joints per 3 m (10 ft) in brittle units.

Cutoff trench excavation during reconstruction revealed solution channels (Figure 12) along joints more than 21 m (70 ft) deep in the foundation. Also revealed were soft zones where cement, mostly gypsum, had been leached leaving behind erodible uncemented silt grains. These features usually were associated with prominent joints or shear zones.

Figure 12. Joint in Unit 5 enlarged by erosion and dissolving of gypsum.

Solubility

During the predesign subsurface foundation investigation, both thickness of gypsum lenses and percentage of gypsum in each 3 m (10 ft) interval were estimated. The highest reported percentage for any interval was 7.6 percent and the predominant lens thickness was determined to be 1.6 mm (0.063 in.). The design considered solubility of gypsum using numerical methods developed by James and Lupton (1978). As stated in the design report (Rollins, Brown and Gunnell, Inc.,1983b), predictive models indicate that a fissure widens at its beginning point and expands in thickness along the fissure depending upon the initial fissure size, solubility of the rock, and length of time of seepage (James and Lupton, 1978, James and Kirkpatrick, 1980). The report concluded that since the predominant gypsum lens thickness was 1.6 mm (0.063 in.) and that since no well defined fissures were observed in lenses, only hairline cracks, not exceeding 0.2 mm (0.008 in.), were to be expected. Theoretical studies (James and Kirkpatrick, 1980) suggest that fissures as small as 0.2 mm (0.008 in.) should experience essentially no change after being subjected to a hydraulic gradient of 0.2 for a 100-year period. It was therefore concluded by the designer that the likelihood of the foundation developing seepage problems was relatively small. The flaw in this conclusion came from an erroneous predictive model resulting from the gross underestimation of gypsum content in the foundation rock and the size of initial fissures. James (1990) suggested that an appropriate model would be fissure flow in which water passes over the surface of solid gypsiferous material. In retrospect, a conservative approach to the review of foundation seepage may have been to perform a reanalysis using the maximum joint openings reported by State-employed geologists in their cutoff trench log. Joints were observed to be open as much as 0.63 cm (0.25 in.). Using that maximum opening size and appropriate seepage velocity, the analysis would likely have predicted unacceptable solution rates for the jointed gypsiferous rock.

Grouting sequences during operations began to seal some seepage paths and locally increased hydraulic gradients in others. James (1990) explained that "water passing over gypsum becomes chemically saturated within a flow path and beyond this zone of saturation, no further dissolution occurs. As flow continues, the zone moves downstream and eventually passes from the exit. At this stage, dissolution

rates accelerate sharply." This could explain the erratic breakthroughs and sudden increases in seepage which were observed.

Results of studies by Morrison-Knudsen Engineers, Inc. (1989) also confirmed James' comments regarding sensitivity of gypsum solubility to hydraulic gradient and flow velocity. The report indicates that for a seepage velocity of 10^{-4} cm/sec (103 ft/yr), a 2 cm (0.87 in.) wide gypsum vein should dissolve at the rate of a few cm per year from an advancing front. If seepage velocities were about 10^{-2} cm/sec (10,340 ft/yr), the gypsum could dissolve at the rate of about 9 m (30 ft) per year. Gypsum dissolution occurs until seepage water reaches a calcium sulfate saturation of about 2,000 parts per million. Hence the dissolution zone moves downstream as greater quantities of unsaturated water attack a gypsum vein. Other soluble salts present in foundation rocks may have aggravated the gypsum solubility problem.

Hydraulic Conductivity

Water pressure testing in angle borings during reconstruction exploration suggested that hydraulic conductivity generally ranged from 10^{-1} to 10^{-6} cm/sec (1,034 to 1 ft/yr) with an overall average of 10^{-4} cm/sec (103 ft/yr; Morrison-Knudsen Engineers, Inc., 1990). Local zones of high water take occurred where the borings crossed open joints, solution cavities, or pervious bedding planes. Average values of low hydraulic conductivity in this foundation are misleading since water conduits with high hydraulic conductivities exist in isolated locations such as along open joints. In general, there was not a detectable decrease in the average hydraulic conductivity to the depth explored (82 m [270 ft]).

DESIGN CONSIDERATIONS

Quail Creek Dike design began in 1982, and final geotechnical and design reports were submitted to the Washington County Water Conservancy District in 1983 (Rollins, Brown and Gunnell, Inc. 1983a and 1983b). The initial exploration consisted of seven vertical holes. The alignment was then shifted upstream to reduce embankment volume, and three additional vertical holes were drilled. From water pressure tests it was concluded that rock below a depth of 3 m (10 ft) was relatively impervious and would not present problems in regard to seepage. As a result of the geotechnical investigation, the following design assumptions and decisions were reached:

1. A cutoff trench to an average depth of 3 m (10 ft) would be adequate to control seepage by cutting off the weathered zone.
2. Grouting would not be effective or necessary in the dike foundation except for the sandstone in the upper left abutment from about station 0+00 to 2+00.
3. Special treatment, such as dental concrete or slush grouting, was not required for rock exposed in the cutoff trench except for the sandstone high in the left abutment.
4. The hogback ridges would be left in place, except for the cutoff trench excavation, and stripping over the foundation to a depth of 30 cm (1 ft) would be adequate.
5. Zone I (silty sand) would be placed to fill potholes, undulations and drainage channels outside of the cutoff trench.
6. Impervious, medium-plasticity clay would be used in Zone II upstream of Zone I to increase the safety against seepage and solution of soluble salts in Zone I materials. A shallow thickness of Zone II was also to be placed on the bottom of the cutoff trench to reduce seepage.
7. A vertical filter was designed just downstream of Zone I based on the design conclusion that the low-plasticity Zone I material must be protected by a satisfactory filter.
8. Gypsum content, as indicated on boring logs, was assumed to average about 3 percent in the Shnabkaib Member. Gypsum was described to be present as laminations which grade laterally and vertically into siltstone.

CONSTRUCTION

Pay quantity cross sections indicated that the cutoff trench actually ranged from 0.6–7 m (2–23 ft) deep and was as narrow as 5.8 m (19 ft). The actual depth of the cutoff was determined by judgment in the field based on quality of the rock exposed. Excavation through the troughs between the hogback ridges were usually substantially less than 3 m (10 ft) in depth (Figure 9). Although the tendency to excavate more from hogbacks and less in troughs smoothed the profile, it left some of the most inferior rock at the dam-foundation contact. The more erodible rock units were located in the troughs, as demonstrated by post

the volume and velocity of seepage near the contact and thus hastened the seepage erosion process.

4. The existence of an extensive joint system with some open joints was not recognized or considered by designers.
5. Defensive design features in the form of filters to prevent internal erosion of the embankment at the foundation contact were not provided.
6. During construction, use of Zone I material for a leveling course between hogbacks from upstream to downstream should not have been permitted, particularly in view of the potential for internal erosion.
7. Remedial grouting during operations did not prove to be a long term solution for this particular foundation as demonstrated by the shifting locations of seepage emergence during grouting and sporadic outbreaks of new seepage after completion of each episode of remedial grouting.

ACKNOWLEDGMENTS

The authors thank R. L. James, Chairman of the Independent Review Team, and R. B. Hall, Directing Dam Safety Engineer, Utah State Engineer's Office for their critical review of this paper.

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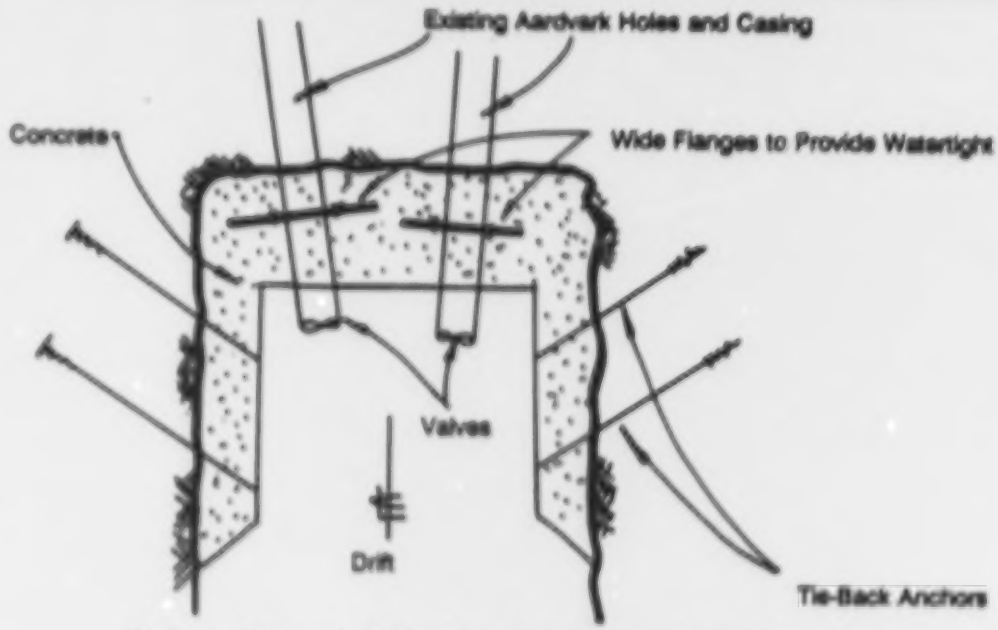
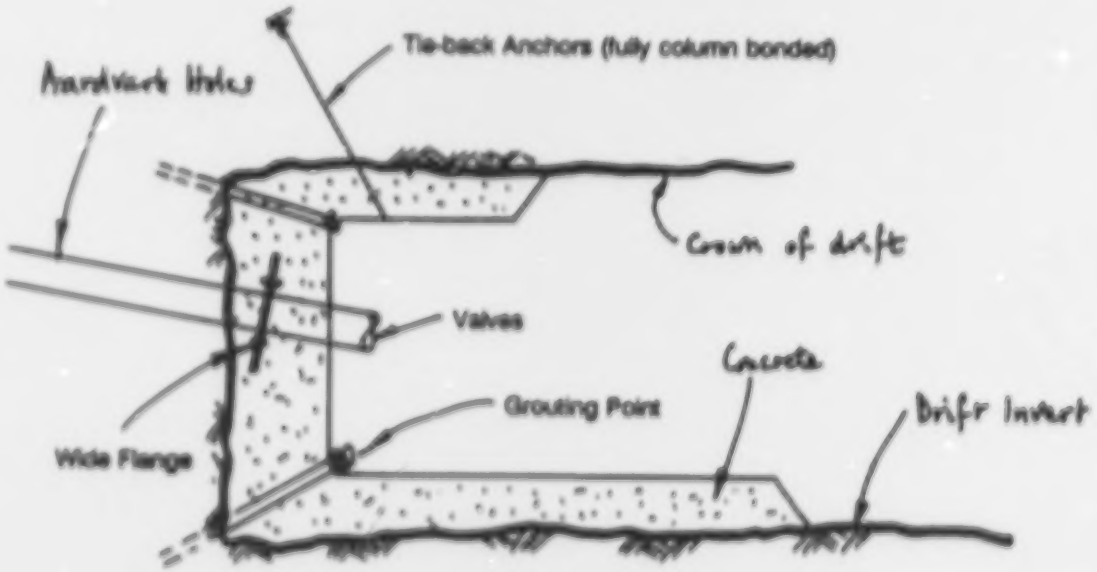
Sketch Diagrams showing Conceptual Arrangement of Concrete Bulkhead for Drainholes in Evaporites

FIGURE 1



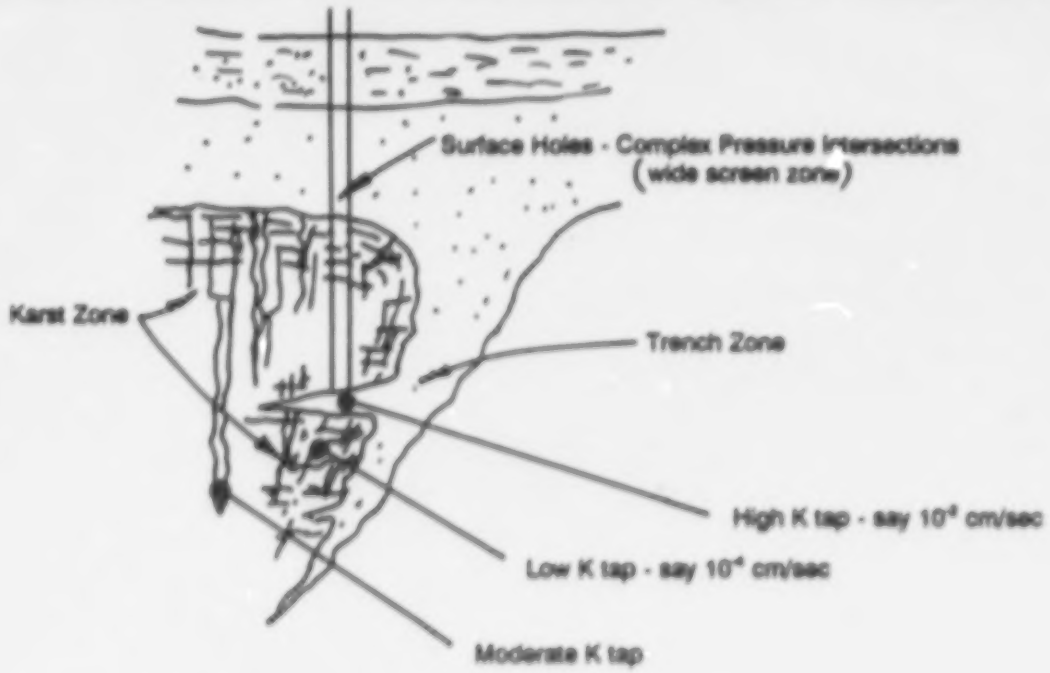
Sketch Plan of Drift showing Concrete Bulkhead Arrangement



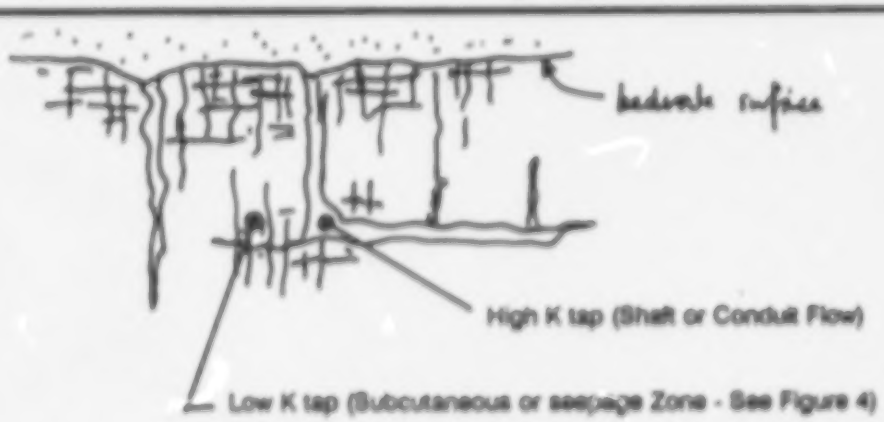
Sketch Section

Sketch Cross-Section and Longitudinal of Trench illustrating Possible Reasons for Pressure Regime

FIGURE 2


Surface Holes - Complex Pressure Intersections
(wide screen zone)
Karst Zone
Trench Zone
High K tap - say 10^{-3} cm/sec
Low K tap - say 10^{-4} cm/sec
Moderate K tap

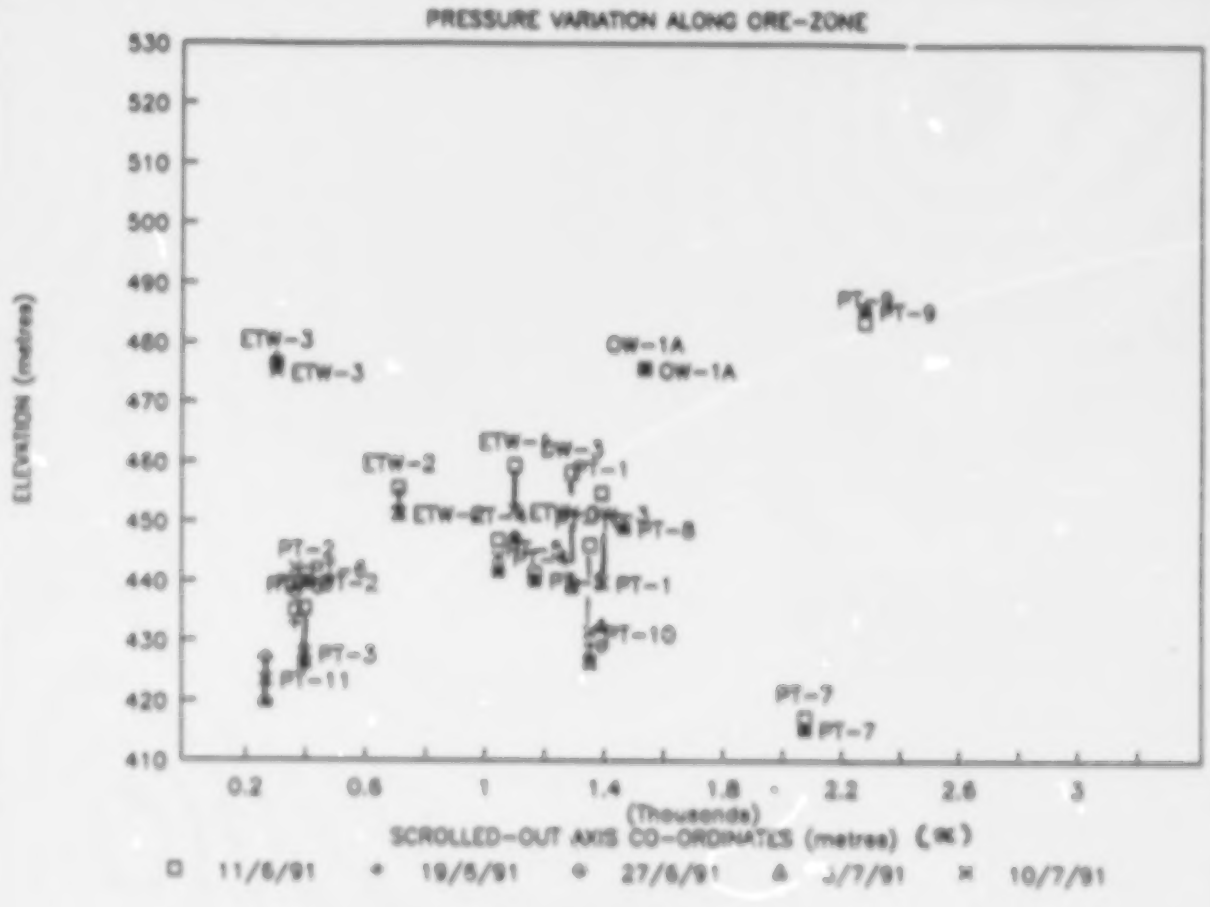
Schematic Cross-Section showing High and Low K zones



Schematic Longitudinal of Hangingwall Karst Zone

Scrolled Out - Longitudinal Section of Mining Zone showing Groundwater Pressure variations along Strike

FIGURE 3



NOTES

1. PRESSURE HEAD DATA SEEMS TO STRONGLY SUGGEST THAT DISTINCT - NON CONNECTED DRAWDOWN CONES EXIST
2. SURFACE WELLS WITH INTERCONNECTED SCREENS ARE MEASURING COMPLEX INTERRELATED WATER LEVELS
3. PRESSURE HEADS IN UNDERGROUND BOREHOLES VERY DEPENDANT ON GEOLOGICAL ENVIRONMENT INTERSECTED
   - IF A HOLE INTERSECTS MAIN CHANNEL OR CONDUIT - WILL LIKELY SEE LOW PRESSURE HEADS
   - IF A HOLE INTERSECTS ADJACENT LOW PERMEABILITY ZONE (ie NOT THE MAIN CONDUIT) - PRESSURE HEADS MAY BE HIGHER

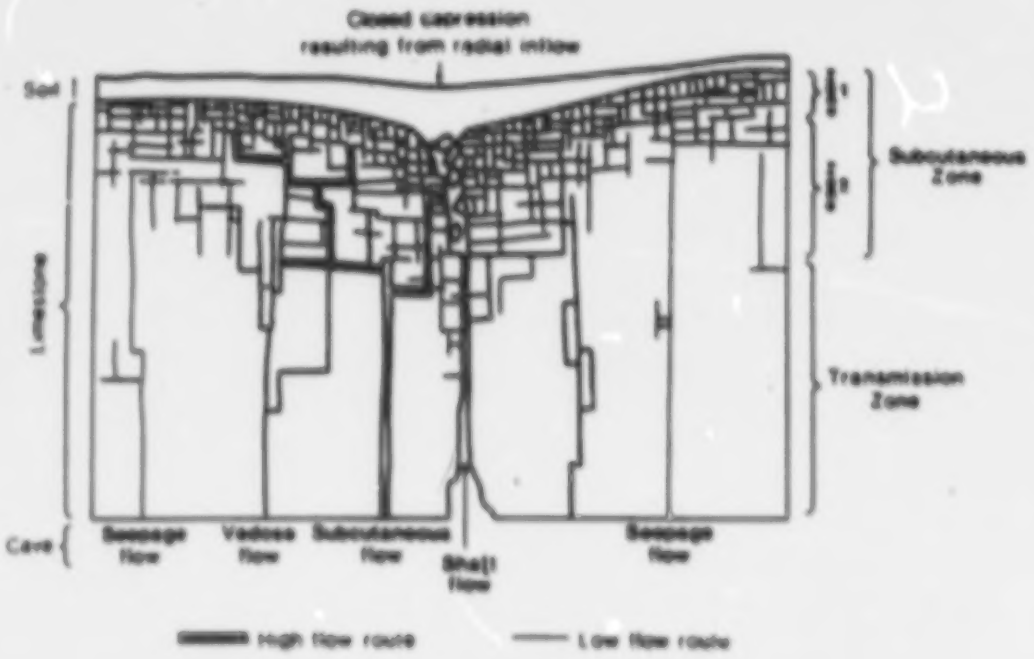
PRESSURE POINTS	Elev.	SC. (m)
PT-1	411.0	1380.0
PT-2	430.7	403.0
PT-3	421.8	366.0
PT-4	441.8	1048.0
PT-5	428.7	1188.0
PT-6	427.3	968.0
PT-7	412.0	2576.0
PT-8	438.0	1465.0
PT-9	471.8	2380.0
PT-10	408.0	1360.0
PT-11	420.0	246.0

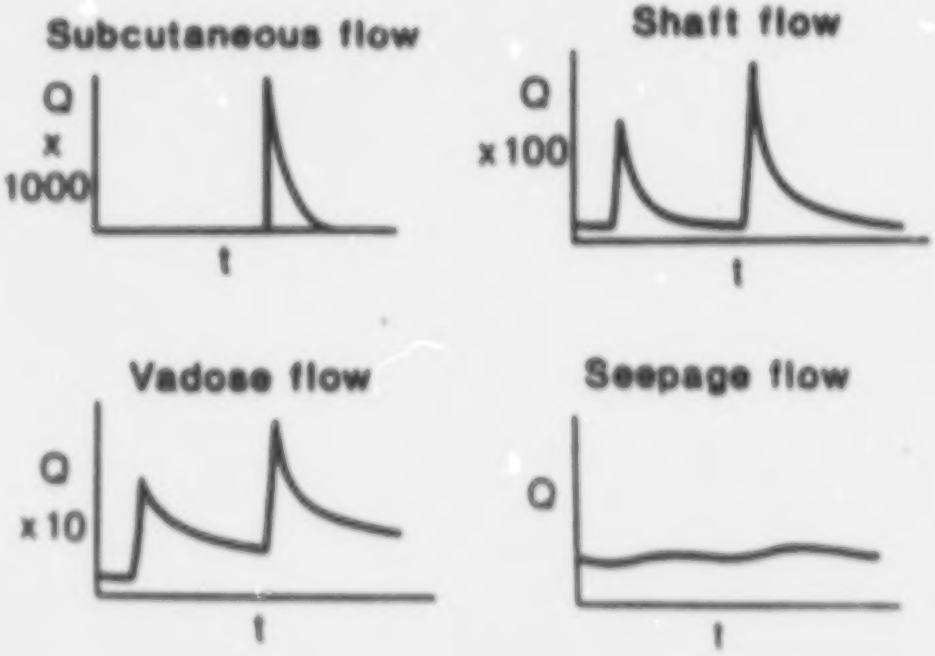
MONITORING WELLS*	SC. (m)
OW-1A	1588.0
ETW-1	1186.0
ETW-2	716.0
ETW-3	300.0
OW-4	1280.0

* ZERO FOR SCROLLED-OUT AXIS IS WEST EDGE OF MINE WORKINGS

Conceptual Arrangement of Flow Conduits and Fracturing in Classic Karst Environments

FIGURE 4





Hydrographs of Flow from Two Sequential Hypothetical Rainfall Events

GOLDER ASSOCIATES LIMITED
2180 Meadowvale Blvd.
Mississauga, Ontario
L5N 5S3

Fax: 416-567-6561

Tel: 416-567-4444

FACSIMILE TRANSMISSION

Date Sent: June 21, 1991

TO: Guys River Mine
FAX NO: (902) 883-9245

ATTN: Brian Mickn/Tom Lamb

FR: Trevor Carter

RE: AARDVARK DRILL TARGETS

JOB NUMBER: 911-1721

Total number of pages (including this cover page): 3

MESSAGE:

We've now had time to do some calculations for the various holes tested last week. The table below gives the relatively "solid" results. We are still short of data on some of the holes as evident from the blanks. However these have been guestimated on the attached separate table.

	Hole #	Test #	Location	Collar Elevation (m)	Max Pressure Head			Approx.† Hydraulic Conductivity (cm/sec)
					Water-head Elevation	(m)	(psi)	
•	U11840	PT-1	118-2 Sump	411	454.8	43.8	62.3	
•	U625	PT-2	825 Ramp	430.7	441.2	10.5	14.9	1×10^{-3}
•	Longhole	PT-3	808 X-Cut	421.5	435.5	14.0	19.9	
	U597	PT-4	848 Ramp	441.9	446.8	4.9	7.0	6×10^{-4}
	U641	PT-5	851 Decline	428.7	441.3	12.6	17.9	2.5×10^{-3}
•	U649	PT-6	803 Sump	427.3	435.0	7.7	11.0	6×10^{-5}
·	Longhole	PT-7	101 D	412.0	417.2	5.2	7.4	–
·	D11801	PT-8	119 Area	428.0		–	–	–
·	U563	PT-9	210 Drift	471.8	483.7	11.9	16.9	8×10^{-5}
•	U118-26	PT-10	118 Sump	409.0	446.1	37.1	52.7	
TARGET SITES	† based on an assumed zone inflow corresponding with the trench margins.							

• Primary Target Areas
· Secondary Target Areas

From a review of the preliminary data that we have gathered (see attached table as well as summary tabulation given above), the best target sites are those that exhibit:-

- the highest locked-in pressure,
- the highest hydraulic conductivity, and
- the lowest access level elevation.

On this basis the prime target zone is definitely the 118 area and those locations where the hydraulic conductivities are high and the collar elevations of the holes are lowest.

The 210 drift while an attractive zone from a permeability viewpoint is at a very high point in the mine, and the water inflow intersections are only on the upholes. This suggests that although water inflows can be improved here, things would be better addressed to attempting to find a "geological" intersection target in the same area at a lower elevation.

I will advise further on what the data means early next week.

Best regards.

TGC/se

P.S. Please send me by fax the latest pressure head data on these holes and any others that are now gauged.

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FACSIMILE TRANSMISSION

Date Sent:	June 14, 1991
TO: FAX NO:	Westminer (Canada) Ltd. (902) 583-9245
ATTN:	Brian Micke/Tom Lamb
FR:	Trevor Carter/Rob Blair
RE:	GAYS RIVER DEWATERING
JOB NUMBER:	911-1721

Total number of pages (including this cover page): 5

MESSAGE:

a) UNDERGROUND

Some further points to ponder:

- Think about using Aardvark louvred stainless steel screens for insertion in 118 production holes, but only after drilling some new Aardvark holes to take "load" off the two producing wells. (This screen is stronger than the wire screens and thus it should be possible to push it up the production holes).

- I have sketched out on Figure 1 a "worst case" drift infill filter which will still allow access to the holes for cleanout and which would stop all fines migrating out of system. It is basically a "dam drainage filter design" (see gradations on Figure 2 - which are designed to stop migration of clay from a dam core (Zone 1)). As we discussed it would only be necessary to go to these lengths in the situation that we start to experience large influxes of fine sediment. I don't think this is particularly likely if we can keep control of water pressure conditions. However, it is as well to have these ideas up our sleeve.

b) SURFACE

Further to discussions yesterday regarding groundwater inflow through overburden:

- Influence Zones - Need to evaluate zones of groundwater level drawdown toward the mine within the sand overburden. This is necessary to establish contribution and to see if we can engineer a solution. Gradient must be defined to determine the potential influence (area and influx) of surface groundwater contribution. This will require

piezometers installed in the overburden within an area extending approximately 1000 to 2000 m. outwards from the centre of the mine.

- **Flow Paths** - Need piezometers in the sand filled valleys around the mine to determine groundwater levels and hence directions of groundwater flow. Can then assess potential groundwater contributions to the mine from various areas. Areas that need checking include the Gays River channel, the valleys extending eastward and southward from the river toward the portal and the two valleys extending northward from the tailings basin area.

- **Gays River Contribution** - With the mine water being discharged back into the system up river from the mine - we are possibly creating a complete circuit. If the water was discharged downstream of the mine - perhaps the river flow would measureably decrease!

- **Vertical Hydraulic Gradients** - Need at least two piezos in each hole, one deep and one shallow. Can then establish vertical hydraulic gradients; ie. the change in water level with depth. Where groundwater is moving laterally through the aquifer, there will be little difference in water levels between the shallow and deep piezometers. Where groundwater is moving downward there will be a comparative large (steep) water level difference between the shallow and deep piezometers.

- **Sinkholes** - The relatively new (and 1980's era) sinkholes are obvious indicators of steep vertical hydraulic gradients. They reflect points of groundwater infiltration. Accordingly they are obvious piezometer installation targets.

- **Piezometer Locations** - Require piezometers along the Gays River to assess groundwater contribution within the potential zone of mine dewatering influence, specifically areas of trench crossings and sinkhole development (both geological and new origin) (suggest 10 hole locations).

- Require piezometers to assess groundwater contribution from the sand infilled valleys extending toward the mine areas, eastward from the river beneath the administration building and northward from the tailings basin (suggest 5 locations).

- Require piezometers in overburden above portal and 118 sump areas. (As this area is likely largely dewatered suggest 1 deep piezometer only.)

- Require piezometers to assess groundwater contribution from Taylor Brook Valley (suggest 2 locations).

- Require piezometers to assess groundwater contribution from Black Brook sinkhole area (suggest 1 location).

- **Geophysics** - Ground penetrating radar surveys are considered to be the most suitable for assessing groundwater surface topography in the sand filled valleys. The method is also suitable for defining bedrock to depths of 30 m. and indicating overburden stratigraphy provided there isn't too much clay in the overburden. It also may allow reasonable

definition of the watertable in sand or gravel provided there are some piezometer measurement points to calibrate with.

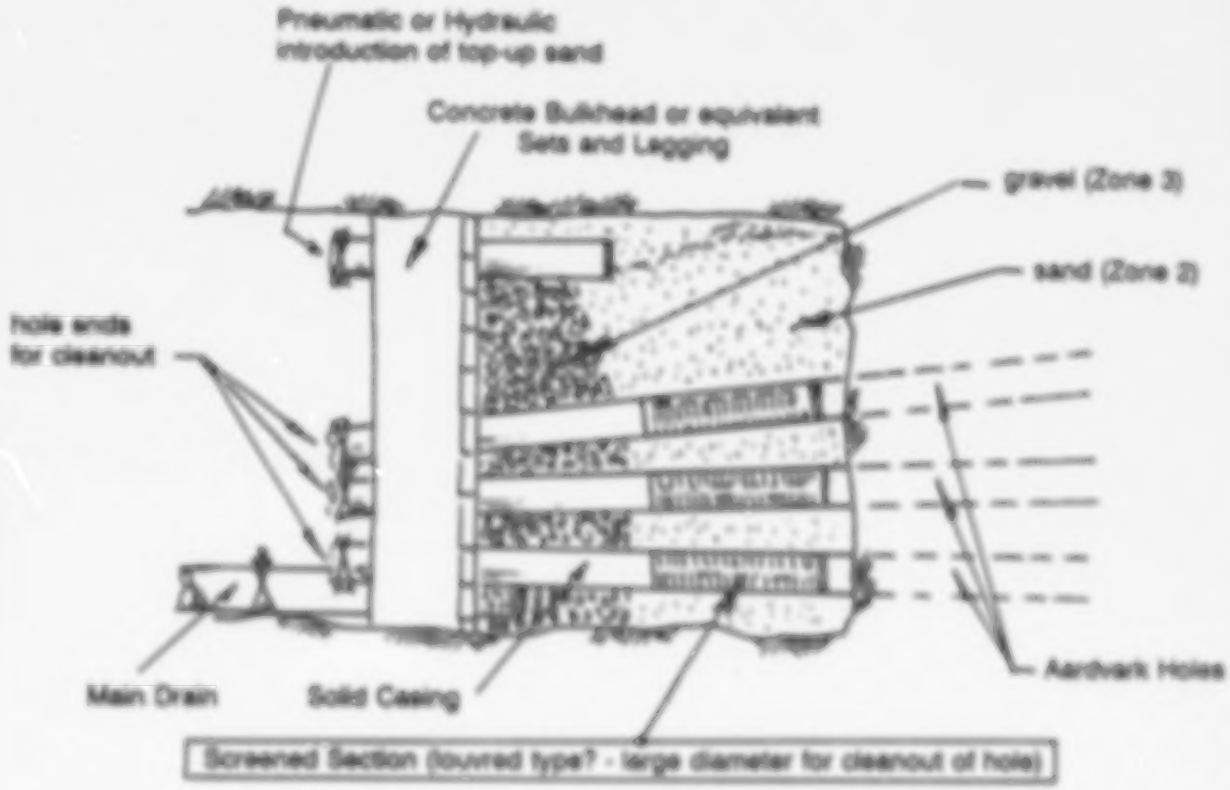
A trial effort of 1 day on site with the radar will determine the suitability of the system to see anything. If successful, a reasonable line spacing (say 100 m.) between the suggested overburden piezometer locations could potentially define localized areas of watertable depression within the overall mine drawdown zone. These localized depressions would potentially indicate "point" sources of mine infiltration and thus aid mapping of potential locations of new sinkhole development. These, then may warrant additional coverage to better define them.

The radar survey may also provide an indication of cavernous areas in the bedrock to depths of 40 m. (provided depth penetration is adequate). Areas of clay overburden in excess of 2 to 3 m. will tend to block radar penetration through.

TGC/RDB/se

Gays River Mine - Aardvark Dewatering Holes
Idea for Possible Secondary Filter (If really needed)

FIGURE 1

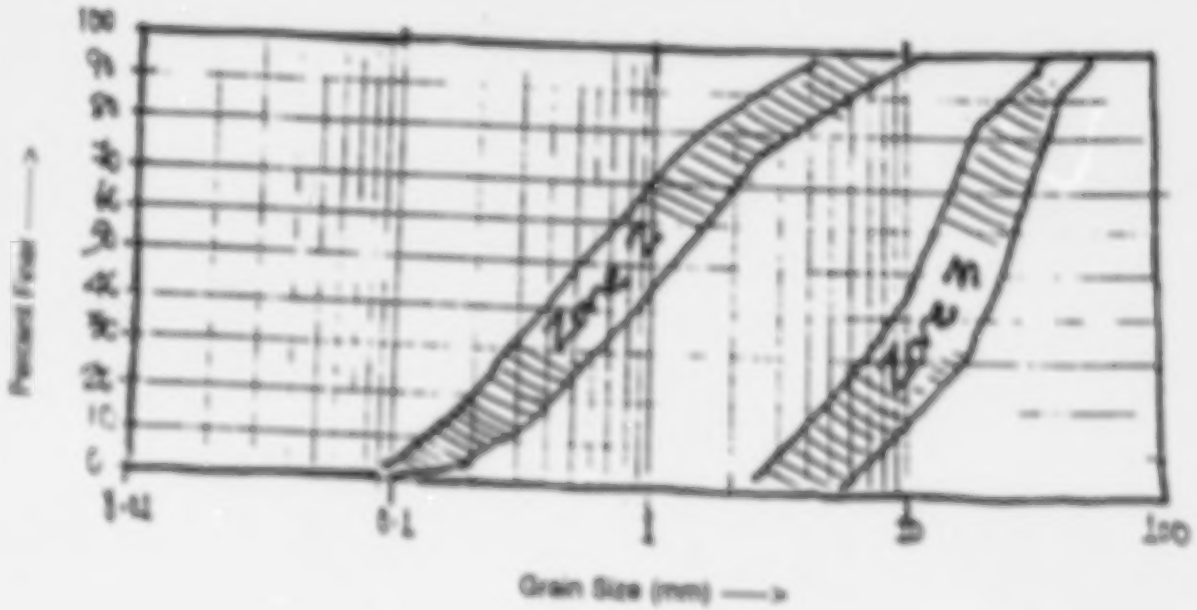

Pneumatic or Hydraulic introduction of top-up sand
Concrete Bulkhead or equivalent Sets and Lagging
gravel (Zone 3)
sand (Zone 2)
hole ends for cleanout
Aardvark Holes
Main Drain
Solid Casing
Screened Section (louvred type? - large diameter for cleanout of hole)

Notes:

1. Zones 2 and 3 filter and drain material using normal concrete sand and concrete coarse aggregate usually will fit within the gradation curves shown on Figure 2. These should filter all fines that likely would come through drain system.

2. Existing Butterfly Valve arrangement could be extended out to beyond the bulkhead

Typical Filter (Zone 2) and Drain (Zone 3) Gradings	FIGURE 2



Notes:

1. Zone 2 Filter Sand - typical grading for most standard concrete sands will fit within this envelope.

2. Zone 3 Gravel Drain - typically gradation of most coarse concrete aggregates will fit within the gradation shown above

APPENDIX B

FIELD DATA FROM PRESSURE AND FLOW MONITORING AT SELECTED SITES WITHIN THE GAYS RIVER MINE COMPLEX

PROJECT: GAYS RIVER

DRILL HOLE DATA

No. 911-1721

LEVEL No.: 118-2 COLLAR ELEVATION: 409.0 ANGLE: 0° (UP) (DOWN)

HOLE No.: PT-1 U11840 STATIC (SHUT IN) PRESSURE: 62.5 (P.S.I.) 43.8 (m) (H_i)

HOLE LENGTH: 23 M MIN. DRAWDOWN PRESSURE: 60 (P.S.I.) 42 (m) (S_r)

DATE	CLOCK TIME	ELAPSED TIME			PRESSURE		DRAWDOWN	RECOVERY (S_r - S)	RESIDUAL DRAWDOWN (S')
	H : Min.	t (mins)	t' (mins)	t/t'	(P.S.I.)	m (H)	(H_i - H)	(H - S_r)	(H_i - H)
11/6/91									
10.0 l/min 15.0 l/min					RECOVERY				
	11:02	1440	0		0	4.2			
		1440.5	0.5	2881	11	7.7			
	11:03	1441	1	1441	20	14			
		1441.5	1.5	961	35	24.5			
	11:04	1442	2	721	54	37.8			
	11:07	1445	5	289	62	43.4		1.4	0.4
	11:12	1450	10	145	62	43.4		1.4	0.4
12/6/91					DRAWDOWN				
	11:23	2901	1461.0		62.5	43.8	0		
25 l/min									
	11:24	2902	1		60	42	1.8		
	11:25	2903	2		60	42	1.8		
	11:28	2906	5		60	43	1.8		
	11:33	2911	10		No appreciable change				

PROJECT: GAYS RIVER

DRILL HOLE DATA

No. 811-1721

LEVEL No.: 625 RAMP COLLAR ELEVATION: 441.04 ANGLE: 1.3° (UP)

HOLE No.: U 639 - PT2 STATIC (SHUT IN) PRESSURE: 15 (P.S.I.) 10.5 (ins) (H_o)

HOLE LENGTH: 49.09 MIN. DRAWDOWN PRESSURE: 2 (P.S.I.) 1.4 (ins) (S_o)

DATE	CLOCK TIME	ELAPSED TIME			PRESSURE		DRAWDOWN	RECOVERY (S_o - S)	RESIDUAL DRAWDOWN (S')
	H : Min.	t (mins)	t' (mins)	t/t'	(P.S.I.)	(ins) (H)	(H_o - H)	(H - S_o)	(H_o - H)
11.6.91									
5.7 l/min									
3.9 l/min									
					RECOVERY				
	12:08	1440	0		0	0		--	
		1440.5	0.5		0	0		--	
	12:11	1443	3		0	0		--	
	12:18	1450	10		< 4	< 2.8		1.4	7.7
	12:45	1477	37		6	4.2		2.8	6.3
	15:16	1628	188		13	9.1		7.7	1.4
12.6.91					DRAWDOWN				
44 l/min	13:31	2963	1523.0		15	10.5	0		
		2963.5	0.5		5	3.5	7.0		
	13:33	2965	2		2	1.4	9.1		

PROJECT: GAYS RIVER

DRILL HOLE DATA

No. 911-1721

LEVEL No.: 808 x-Cut (Aardvark Drill Location)

COLLAR ELEVATION: 421.8

ANGLE: 1.0° (UP) (DOWN)

HOLE No.: PT-3 Longhole

STATIC (SHUT IN) PRESSURE: 30 (P.S.I.) 14 (m) (H_s)

HOLE LENGTH: 17.7 m

MIN. DRAWDOWN PRESSURE: 14 (P.S.I.) 9.8 (m) (S_s)

DATE	CLOCK TIME	ELAPSED TIME			PRESSURE		DRAWDOWN	RECOVERY (S_s - S)	RESIDUAL DRAWDOWN (S')
	H : Min.	t (mins)	t' (mins)	t/t'	(P.S.I.)	m (H)	(H_s - H)	(H - S_s)	(H_s - H)
11/6/91									
60 l/min					RECOVERY				
	13:40	1440	0		0	0			
		1440.25	0.25		0	0			
		1440.5	0.5		6	4.2		–	–
	13:41	1441	1.0		11	7.7		–	–
	13:42	1442	2.0		14	9.8		0	4.2
	13:45	1445	5.0		14	9.8		0	4.2
	13:50	1450	10.0		15	10.5		0.7	3.5
12/6/91					DRAWDOWN				
57 l/min	12:55	2035			20	14	0		
		2035.5	0.5		16	11.2	2.8		
	12:56	2036	1.0		16	11.2	2.8		
	12:57	2037	2.0						
	12:58	2038	3.0		14	9.8	4.2		
	13:01	2041	6.0		14	9.8	4.2		
	13:06	2046	11.0		14	9.8	4.2		
					RECOVERY				
	13:12	2052	0		0	0		–	–
		2052.5	0.5		7	4.9		–	–
	13:13	2053	1.0		14	9.8		0	4.2
	13:14	2054	2.0		16	11.2		1.4	2.8
	13:17	2057	5.0		16	11.2		1.4	2.8
	13:22	2062	10.0		17	11.9		2.1	2.1

PROJECT: GAYS RIVER

DRILL HOLE DATA

No. 911-1721

LEVEL No.: 848 RAMP (Mudflow Area)

COLLAR ELEVATION: 441.88

ANGLE: 1.0° (UP)

HOLE No.: US97-PT4

STATIC (SHUT IN) PRESSURE: 7 (P.S.I.) 4.8 (m) (H_s)

HOLE LENGTH: 23.47

MIN. DRAWDOWN PRESSURE: 2 (P.S.I.) 1.4 (m) (S_d)

DATE	CLOCK TIME	ELAPSED TIME			PRESSURE		DRAWDOWN	RECOVERY (S_r - S)	RESIDUAL DRAWDOWN (S')
	H : Min.	t (min)	t' (min)	t/t'	(P.S.I.)	m (H)	(H_s - H)	(H - S_d)	(H_s - H)
11/6/91									
5.5 l/min					RECOVERY				
	14:14	1440	0		0	0		--	--
		1440.25	0.25		0	0		--	--
		1440.5	0.5		3	2.1		0.7	2.8
	14:15	1441	1		4	2.8		1.4	2.1
	14:16	1442	2		4	2.8		1.4	2.1
	14:19	1445	5		5	3.5		2.1	1.4
	14:24	1450	10		6	4.2		2.8	0.7
					DRAWDOWN				
12/6/91	13:56	2882			7	4.0	0		
7.5 l/m	13:57	2883			6	4.2	0.7		
	13:58	2884			5	3.5	1.4		
	14:01	2887			2	1.4	3.5		

PROJECT: GAYS RIVER

DRILL HOLE DATA

No. 911-1731

LEVEL No.: 651 DECLINE COLLAR ELEVATION: 439.69 ANGLE: 20° (UP)

HOLE No.: U 641-PTb STATIC (SHUT IN) PRESSURE: 19 (P.S.I.) 13.8 (m) (H_s)

HOLE LENGTH: 40.64 MIN. DRAWDOWN PRESSURE: 3 (P.S.I.) 2.1 (m) (S_d)

DATE	CLOCK TIME	ELAPSED TIME			PRESSURE		DRAWDOWN	RECOVERY (S_r - S)	RESIDUAL DRAWDOWN (S')
	H : Min.	t (mins)	t' (mins)	t/t'	(P.S.I.)	m (H)	(H_s - H)	(H - S_d)	(H_s - H)
11/6/01									
6.8 l/min					RECOVERY				
	14:41	1440	0		0	0		--	
		1440.25	0.25		0	0		--	
		1440.5	0.5		5.5	2.45		0.35	10.15
	14:42	1441	1.0		4.5	3.15		1.05	9.45
	14:43	1442	2.0		6.0	4.2		2.1	8.4
	14:46	1445	5.0		12.5	8.75		6.65	3.85
	14:51	1450	10.0		16.0	11.2		9.1	1.4
	15:01	1460	20.0		17.5	12.25		10.15	.35
12/6/01					DRAWDOWN				
75 l/min.	13:45	2824	0		19.0	12.6	0		
		2824.5	0.5		15.0	10.5	2.1		
	13:46	2825	1.0		12.0	8.4	4.2		
	13:47	2826	2.0		5.0	3.5	9.1		
	13:50	2829	5.0		3.0	2.1	10.5		

PROJECT: GAYS RIVER

DRILL HOLE DATA

No. 911-1721

LEVEL No.: 803 SUMP COLLAR ELEVATION: 427.28 ANGLE: 3° (UP)

HOLE No.: U 849 - PT8 STATIC (SHUT IN) PRESSURE: 11 (P.S.I.) 7.7 (m) (H_s)

HOLE LENGTH: 38.10 MIN. DRAWDOWN PRESSURE: 6 (P.S.I.) 4.2 (m) (H_d)

DATE	CLOCK TIME	ELAPSED TIME			PRESSURE		DRAWDOWN	RECOVERY (H_s - h)	RESIDUAL DRAWDOWN (h')
	Hr : Min.	t (min)	t' (min)	t/t'	(P.S.I.)	m (H)	(H_s - H)	(H - H_d)	(H_s - H)
11/6/91					8 Holes in area - 150 gpm total flow				
40 l/min									
	15:54	1440	0		0	0			
		1440.25	0.25		4	2.8			
		1440.5	0.5		4	2.8			
	15:56	1442	2.0		5	3.5			
	15:59	1445	5.0		6	4.2			
	16:04	1450	10.0		6	4.2			
					DRAWDOWN				
12/6/91	12:16	2662	0		11	7.7	0		
52 l/min		2662.5	0.5		10	7.0	.7		
	12:17	2663	1		10	7.0	.7		
	12:19	2665	3		9	6.3	1.4		
	12:21	2667	5		8	5.6	2.1		
	12:26	2672	10		7	4.9	2.8		
RECOVERY	12:36	2682	20.0		6	4.2	3.5	0	3.5
	12:36:30	2682.5	.5		7	4.9		0.7	2.8
	12:37	2683	1.0 (21)		8	5.6		1.4	2.1
	12:38	2684	2.0 (23)		10	7.0		2.8	0.7
	12:41	2687	5.0 (26)		11	7.7		3.5	0

PROJECT: GAYS RIVER

DRILL HOLE DATA

No. 911-1721

LEVEL No.: 1010

COLLAR ELEVATION: 412
(4 m above invert of dots)

ANGLE: 0° (UP)

HOLE No.: PT7 Longhole

STATIC (SHUT IN) PRESSURE: 7.5 (P.S.I.) 5.25 (m) (H_s)

HOLE LENGTH: 32

MIN. DRAWDOWN PRESSURE: 2 (P.S.I.) 1.4 (m) (S_r)

DATE	CLOCK TIME	ELAPSED TIME			PRESSURE		DRAWDOWN	RECOVERY (S_r - Si	RESIDUAL DRAWDOWN (S')
	H : Min.	t (min)	t' (min)	t/t'	(P.S.I.)	m (H)	(H_s - Hi)	(Hi - S_r)	(H_s - Hi)
11 1/91									
44.4 l/min	Muddy Water				RECOVERY				
	17:00	1440	0		0	0		–	–
		1440.25	0.25		4	2.8		1.4	2.45
		1440.5	0.5		4	2.8		1.4	2.45
	17:01	1441	1.0		5	3.5		3.1	1.75
	17:02	1442	2.0		5	3.5		2.1	1.75
	17:05	1445	5.0		5	3.5		2.1	1.75
	17:10	1450	10.0		5	3.5		2.1	1.75
12/6/91					DRAWDOWN				
44.4 l/m	11:48	2568	0		7.5	5.25	0		
		2568.5	0.5		7.5	5.25	0		
	11:48	2568	1.0		5.0	3.5	1.75		
	11:50	2570	2.0		3.0	2.1	3.15		
	11:53	2573	5.0		2.0	1.4	3.85		
					VALVE SHUT AGAIN				

PROJECT: GAYS River

DRILL HOLE DATA

No. 011-1721

LEVEL No.: 119C Clean-out Drift COLLAR ELEVATION: 438 ANGLE: 20° (UP) (Down)

HOLE No.: PTB - D11801 STATIC (SHUT IN) PRESSURE: – (P.S.I.) – ins (H_s)

HOLE LENGTH: 33 m MIN. DRAWDOWN PRESSURE: – (P.S.I.) – ins (S_d)

DATE	CLOCK TIME	ELAPSED TIME			PRESSURE		DRAWDOWN (ins)	RECOVERY (S_r - S)	RESIDUAL DRAWDOWN (S')
	H : Min.	t (mins)	t ' (mins)	t/t '	(P.S.I.)	ins (H)	(H_s - H)	(H - S_d)	(H_s - H)
2.2 Lines	17:27				Very slow				
					recovery				

PROJECT: GAYS RIVER

DRILL HOLE DATA

No. 811-1721

LEVEL No.: 201 COLLAR ELEVATION: 471.81 ANGLE: 20° (UP)

HOLE No.: U 963-PT9 STATIC (SHUT IN) PRESSURE: 17 (P.S.I.) 11.9 (m) (H_s)

HOLE LENGTH: 42.99 MIN. DRAWDOWN PRESSURE: 2 (P.S.I.) 1.4 (m) (H_d)

DATE	CLOCK TIME	ELAPSED TIME			PRESSURE		DRAWDOWN (m)	RECOVERY (H_s - h_1)	RESIDUAL DRAWDOWN (H')
	H : Min.	t (mins)	t' (mins)	t/t'	(P.S.I.)	m (H)	(H_s - H)	(H - h_1)	(H_s - H)
11/6/91									
17.8 Litres	10:15	1440	0		hole makes gas				
		1440.25	.25		0	0			
		1440.5	.5		<7	<4.9			
	10:17	1441	1.0		<7	<4.9			
	10:18	1442	2.0		7	4.9		3.5	7.0
	10:19	1443	3.0		10	7		5.0	4.9
	10:24	1448	8.0		13	9.1		7.7	2.8
	10:26	1450	10.0		15	10.5		8.1	1.4
12/6/91					DRAWDOWN				
124 Litres	10:48	2422			17	11.9	0		
		2422.5			8	5.6	6.3		
	10:49	2423			7	4.9	7.0		
	10:50	2424			6	4.2	7.7		
	10:53	2427			3	2.1	9.8		
	10:58	2442			2	1.4	10.5		

PROJECT: GAYE RIVER

DRILL HOLE DATA

No. 011-1721

LEVEL No.: 118 SUMP

COLLAR ELEVATION: 409

ANGLE: 15° (UP)

HOLE No.: PT 10 UP 118 20

STATIC (SHUT IN) PRESSURE: 53 (P.S.I.) 37.1 (ins (H$_s$))

HOLE LENGTH: 10.63

MIN. DRAWDOWN PRESSURE: 51 (P.S.I.) 35.7 (ins (S$_d$))

DATE	CLOCK TIME	ELAPSED TIME			PRESSURE		DRAWDOWN (ins)	RECOVERY (S_s - S)	RESIDUAL DRAWDOWN (S')
	H - Min.	t	t'	t/t'	(P.S.I.)	ins (H)	(H_s - H)	(H - S_d)	(H_s - H)
12/6/91									
50 Umin	11:14	0	0		53	37.1	0		
		.5	.5		51	35.7	1.4	0	1.4
	11:15	1.0	1.0		52	36.4		0.7	0.7
	11:16	2.0	2.0		52	36.4		0.7	0.7
	11:18	5.0	5.0		52	36.4		0.7	0.7

APPENDIX C

MEMORANDUM ISSUED ON 20TH JUNE, 1991 RELATING TO STOPE STABILITY CONSIDERATIONS (SOIL MECHANICS ASPECTS)

GOLDER ASSOCIATES LIMITED
2180 Meadowvale Blvd.
Mississauga, Ontario
L5N 5S3

Tel: 416-567-6664 Fax: 416-567-6561

FACSIMILE TRANSMISSION

Date Sent:	June 20, 1991
TO: FAX NO:	Wrenmines (Canada) Ltd. (902) 883-9345
ATTN:	Brian Mickie/Tom Lamb
FR:	Trevor Carter/Dennis Becker
RE:	SOIL MECHANICS ASPECTS OF STOPE HANGINGWALL STABILITY AT GAYS RIVER MINE
JOB NUMBER:	911-1721

Total number of pages (including this cover page): 5

MESSAGE:

Further points/thoughts for your considerations relating to soils aspects:

1. Failure Behaviour of Trench Infill Materials

   - Failure/inflow mechanisms can be classified into two main groups, as follows:

     - Flow of cohesionless soil (ie. silty/sandy) under high piezometric head
     - Plastic flow/shear failure of cohesive (ie. clayey) soils. For this to happen the trench infill material need not be clayey for the entire zone of influence, clayey material can form an interface between the face opening and sandy material, for example.

   - Ground control to minimize flow potential of cohesionless soil can be achieved by dewatering. Experience has shown that face/opening soil stability can be realized with proper and adequate dewatering implemented prior to mining. (This can be monitored with piezometers in the trench zone.)

   - Plastic flow of clayey soils is controlled by its undrained shear strength which is not affected by dewatering operations in the short term (ie. within hours). To obtain an increase in shear strength and hence increase stability, dewatering would need to be implemented months in advance to allow consolidation of the clayey soil to take place (viz. the clayey material will have a significant time lag effect relative to dewatering progress within the adjacent evaporites). In summary, the undrained shear strength will not be increased within a normally acceptable time frame for immediate mining.

Therefore, design of operations must accommodate the current shear strength of the clayey soils.

- The time lag effect on piezometric conditions within silty and sandy soil will be significantly less than for the clayey soils. Currently, it is anticipated that drawdown within the silty and sandy (cohesionless) soils should occur which a short time frame of say days. However to enhance safe mining operations, piezometers will need to be installed at strategic locations to confirm actual dewatering response in the trench material. Tips should be installed prior to underground opening development in an area (see Figure 1).

- Preliminary calculations for face opening stability in clayey soils against plastic flow/shear failure (based on 6 m high (max) stope face stability considerations) indicate that an undrained shear strength of at least 220 kPa is required for stability at an overburden depth of 60 m. For greater depths, higher strength values will be necessary.

- The operational undrained shear strength of the on-site clayey soils is not currently well-defined. Present information suggests strength values of about 200 kPa. Although localized softer zones also reportedly exist. Therefore, the inferred existing strength is only borderline and corresponds to only marginal stability; complete plastic flow/shear failure cannot be ruled out at this time. Some detailed soil mechanics analysis and testing has to be gathered to evaluate overburden face stability.

- Additional information to better define soil strength characteristics can be gained by taking tube samples (± 70 mm diameter) of the clayey soils. These should be obtained for specialized strength-deformation testing such as undrained triaxial compression with pore-water pressure measurements and consolidation (oedometer) tests. The triaxial test results will provide information on strength and brittleness (ie. strain softening behaviour), while the oedometer test results will provide information on stress history, strength, compressibility and hydraulic conductivity characteristics of the clayey material - these will allow strategic planning of stoping layouts (may need rib pillars for example). For cost estimating purposes, an allowance of $4,500 should be made for a specialized geotechnical laboratory testing program. This cost estimate does not include the field costs associated with obtaining the soil samples (see section 3 for overall estimate).

- If the clayey material exhibits classic "brittle" behaviour in soil mechanics terms (ie. exhibits a distinct peak strength and much lower residual strength) then plastic flow/shear failure could be a major problem. In other areas, it may be more of a deformation consideration with localized sloughing/slumping in softer areas. Considerations should be given to deformation monitoring with simple extensometers and to reducing lengths of openings ie. providing rib pillars (maybe of consolidated fill) or panel mining to enhance overall stability through three-dimensional effects.

2. Instrumentation Installations

- Piezometer installation is very important for the cohesionless silty/sand soils as the strength and behaviour of such soils is very much dependent on effective dewatering. However, the installations must be made prior to opening construction; if not, flows will develop before one gets a chance to install a piezometer from the opening face.

- Useful also to install piezometers within the clayeyer soils to confirm and assess time lag effects. Piezometer installations, in the clayey soils however, are not nearly as important as for the more silty soils. Also, for the sandier and gravelly soils the data we already are collecting will be sufficient. In the clayeyer soils piezometers would help in inferring time lag effects. They could be installed from the working face, but the presence of large particles such as gravel/cobbles/boulders will hinder installation. Further, while push-in types of piezometers are reasonably robust, they will probably be damaged by driving or hammering to install the tips as they were designed to be pushed into softer soils. The installations however could be made by placing the piezometers into prebored anchor holes (say for/or with a splitset) and then sealing the back end of hole by stuffing preassembled bentonite socks into the hole. Further thought will be required to define an appropriate and efficient installation technique, but this should be done.

- As deformation with progressive mining could be a significant problem at deeper levels in the mine (say greater than 60m depth from surface), extensometers to measure deformation response should be considered, particularly for the more clayey soil areas.

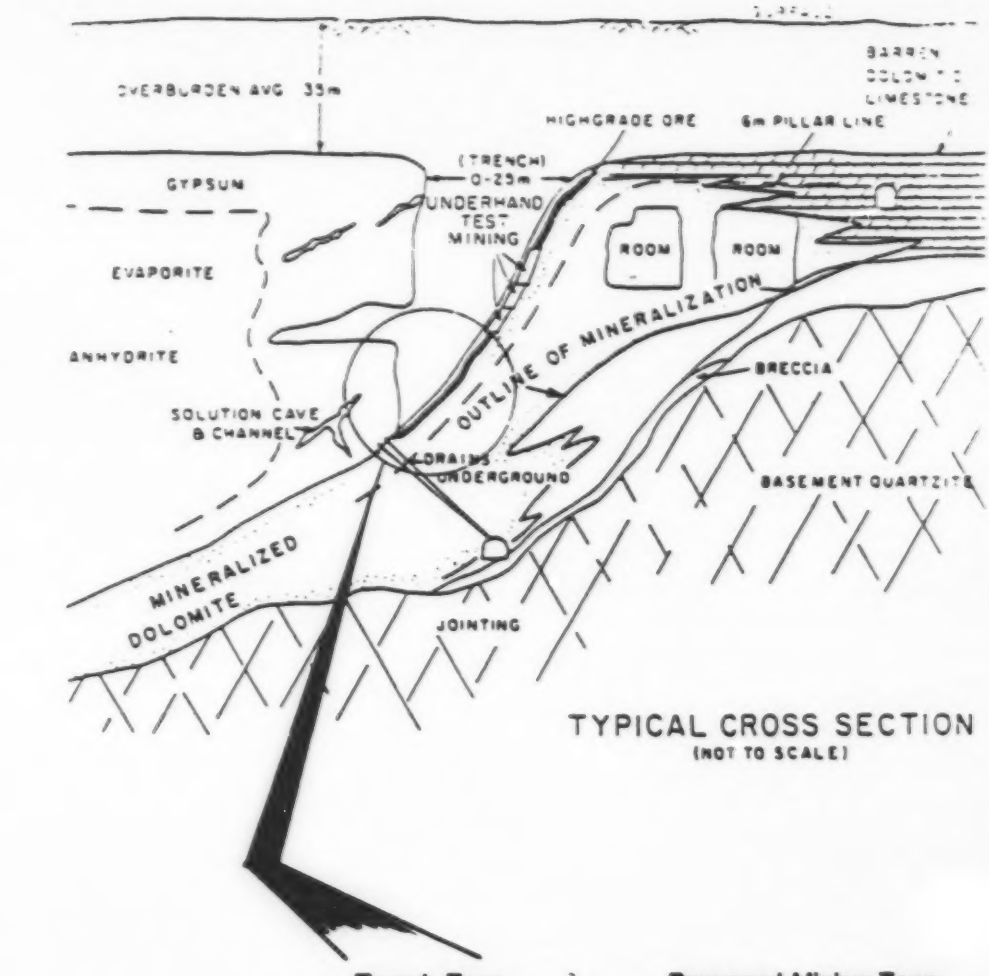
3. Soils Evaluation

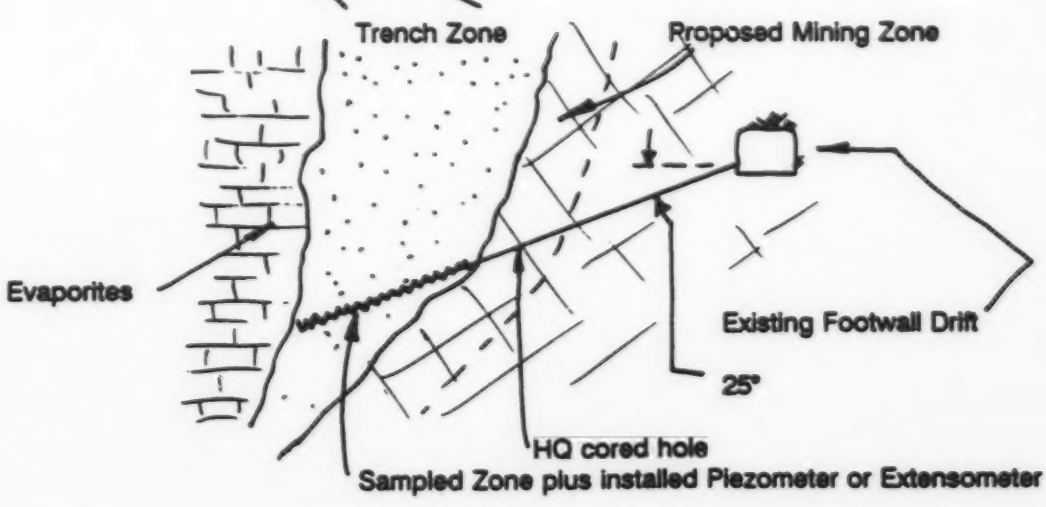
- In order to enhance safe and successful mining operations, some reliable, additional subsurface soil data must be collected prior to mining the deeper parts of the orebody. We need to develop an idea of the distribution of different soil types along the trench, probably plotted onto the overall scrolled out long section. The distribution and piezometric conditions of sandy materials, "pipes", water-bearing granular channels, soft clayey soils and the like must be known so that strategic planning of stoping arrangements (panel mining for instance) can be implemented.

- Drilling and sampling the deep trench soils from the ground surface is impracticable from a cost viewpoint as the holes would need to be very deep (up to 100 m or more). Such boreholes are very tedious, time-consuming and expensive to carry out successfully. Consideration should be given to underground drill locations using 25° downward inclined holes (Figure 1). Such holes, if cored at HQ size into the trench material would yield sufficiently good samples for testing and probably also for extensometer or piezometer installation purposes. A specialist drilling contractor would need to be retained to drill the HQ cored holes under full-time supervision of geotechnical/hydrogeological personnel. For field work cost estimating purposes, a cost of about $100/ft. can be assumed for the drilling and supervision of these type of underground holes.

Gays River Mine Site
Possible Layout of Soil Mechanics Instrumentation for Stability and Pore Pressure Monitoring

FIGURE 1





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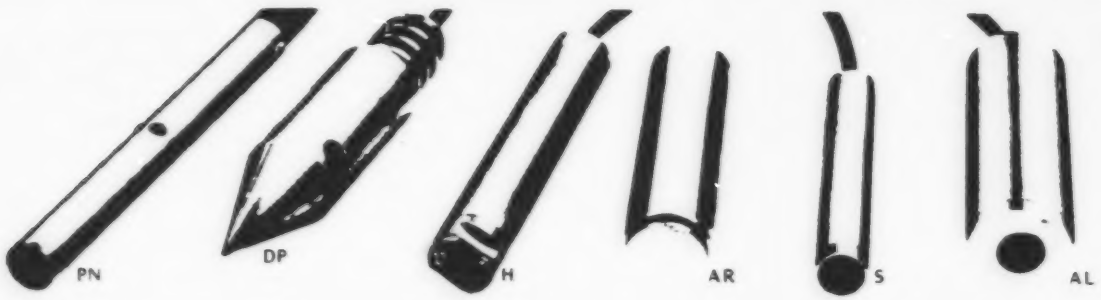
GEOTECHNICAL
INSTRUMENTATION

Vibrating Wire Piezometers and Pressure Transducers



Geokon Vibrating Wire Piezometers and Pressure Transducers are designed to measure fluid pressures (e.g. ground water elevations, pore pressures) in boreholes, embankments, foundations, pipelines, wells and pressure vessels.

Features
- Long-term stability in difficult environments
- Hermetically sealed
- High resolution
- Stainless steel construction
- Not affected by long cable lengths

The transducer employs a sensitive diaphragm, coupled to a vibrating wire, that converts the fluid pressures to be measured into an equivalent frequency signal. Transmitted through a cable to the readout location, the frequency signal is conditioned and displayed on a portable readout box. Alternatively, Dataloggers can be used to record the data automatically. (Low cost battery operated loggers for remote, unattended data collection are available).

Several different models and options are available. The standard sensor is dimensionally small enough to be installed into fills, embankments, boreholes, standpipes or observation wells. A drive-in version is available for pushing directly into soft ground on the end of drill rods. Another model is for use in typical pressure transducer applications where a hydraulic pressure connection is required to permit easy coupling to pressurized tanks, pipelines, earth pressure cells etc.

Construction: Geokon piezometers utilize the latest developments in vibrating wire pressure transducer technology. The transducer is fabricated entirely from stainless steel components, selected to minimize temperature effects and welded together to create a hermetically sealed chamber for the vibrating wire element. The ends of the vibrating wire are clamped using a patented swaging technique that ensures high stability while allowing miniaturization of the components.

Electromagnetic coils located close to the wire are used to both pluck the wire and to convert the wire vibrations so produced into an electrical output current whose frequency is identical to the natural resonant frequency of the wire. Changing pressure causes deflection of the diaphragm thereby altering the tension of the wire and its resonant frequency of vibration. Thus for each pressure there is a corresponding frequency output.

Electrical cable: 4-conductor cable is standard. Two of the conductors are available for use with the built-in thermistor (or VW temperature gage) if temperatures are to be measured. The stanoard cable has a thick PVC outer jacket. Optional direct burial cable is armored and gel-filled. It should be noted that lead wire resistance changes brought about by water penetration, temperature variations and contact resistance do not affect the frequency of the output signal. This factor, coupled with excellent zero stability, makes the Geokon Vibrating Wire Transducer preferable to conventional strain gage transducers for long-term measurements in difficult environments.

In geotechnical environments it is necessary to use a filter stone to prevent soil particles from impinging directly on the diaphragm. The standard filter stone is made from sintered stainless steel with a 50 micron pore size. In partially saturated soils, where air is to be excluded from the piezometer cavity, a high air entry filter stone may be specified. Note that negative pressures (suction) can be measured.

geokon

For further information contact:
Solinst Canada Ltd. *FAX: (416) 873-1992* *TEL: (416) 873-2255*
The Williams Mill, 515 Main Street, Glen Williams, Ontario. L7G 3S9

Solinst

MODELS AVAILABLE

Model 4500S STANDARD MODEL

This model is designed for direct burial in embankments, fills, etc. Also for installation inside boreholes or observation wells. Its small size allows for its installation in conventional standpipe piezometer riser pipe (minimum I.D. is ¾").

Model 4500DP DRIVE POINT MODEL

This transducer is located inside an EW drill rod coupling with a pointed nose cone attached. When screwed onto the end of EW drill rods the unit can be pushed directly into soft ground with the signal cable located inside the drill rod. This model is recoverable at the end of the job.

Model 4500B/C SMALL DIAMETER MODELS

The Model 4500B is designed to go down a ¾" Sch.80 standpipe. Model 4500C is designed to go down a ½" Sch.80 standpipe.

Model 4500PN VIBRO/PNEUMATIC MODEL

A pneumatic transducer packaged in tandem with the vibrating wire transducer permits a second evaluation of the measured pressures. The signal cable is supplemented by two pneumatic tubes.

Model 4500AL/ALV LOW PRESSURE MODEL

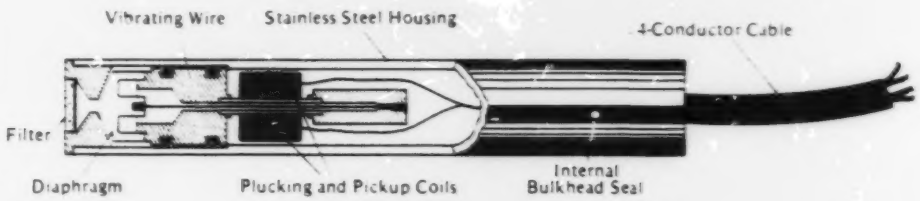
For low pressure ranges (0-5,0-10psi), vented (ALV) and unvented (AL) transducers are used. For long-term (over twelve months) outdoor use the unvented version is recommended with compensation for barometric pressure changes provided by a second unvented transducer acting as a barometer. The low pressure models are ideally suited for the measurement of water levels and stream levels where fluctuations as small as .02inches (0.5mm) can be detected.

Model 4500H PRESSURE TRANSDUCER MODEL

This model is supplied with a ¼" male or female pipe thread fitting to permit the transducer to be coupled directly into hydraulic or pneumatic pressure lines.

Model 4500AR AUTORESONANT MODEL

This model is excited by 12v D.C. and gives a continuous frequency output. For industrial applications the output can be readily adapted to microprocessor based controls or to continuous 0-10v or 4-20mA output. Using the Geokon 4500CPR controller, the transducer can follow dynamic pressure variations of up to 120Hz. These units can be hardened for use in high radiation environments.



Readout Instrumentation

All models can be read using the Geokon Model GK-401 Portable Readout Box. The box displays the reading on a 5-digit liquid crystal display (LCD). The displayed readings can be converted into engineering units using calibration data supplied with the piezometer. Junction boxes and terminal boxes are available to protect cable splices and electrical connectors. Automatic and remote recording can be accomplished using any of the range of Geokon Dataloggers (See the Datalogger data sheet).

GENERAL SPECIFICATIONS

Standard ranges (lbs./sq.in.)	5,10,25,50,100,250,500,1000,5000
Over range	2X rated pressure
Resolution	.025% F.S. minimum
Accuracy	±0.5% F.S. (±0.1% F.S. optional)
Operating temperature	−20° to 150°F (−29° to 65°C)*
Thermal zero shift	<0.02% F.S./°F
Diaphragm displacement	<.001cc at F.S.
Cable	vinyl, 2 pair, 22ga. 0.25in(6.4mm) dia.
Filter	Standard: 50 micron sintered stainless steel High air entry: Alumina approx. 2 microns

*Units for use up to 200°C are available as an option.

WEIGHTS AND DIMENSIONS

Model	D in (mm)	L in (mm)	Wt. lbs(kg)
4500S	0.750(19)	5.250(133)	.26(.12)
4500DP	1.312(33.3)	6.750(171)	1.0(.45)
4500PN	1.000(25.4)	9.125(181)	0.5(.23)
4500B	0.688(17)	5.250(133)	.23(.11)
4500H	1.000(25.4)	5.000(127)	1.0(.45)
4500AR	0.750(19)	8.250(210)	0.5(.23)
4500AL/V	1.500(38)	5.250(133)	1.0(.45)
4500HD	1.000(25.4)	5.000(127)	1.0(.45)
4500C	0.438(11.1)	5.000(127)	.20(.09)

Ordering Information

Specify: 1.Model number.
2.Pressure range.
3.Cable length.
4.Options required.

Optional Accessories

Thermistor
High-entry filter stone.
Gel-filled cable.
Terminal boxes.
Junction boxes.
Lightning protection.

For further information contact us . . . . .

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